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**ENGINEERING AND DESIGN**

# River Hydraulics

**ENGINEER MANUAL**

**DEPARTMENT OF THE ARMY** EM 1110-2-1416
**U.S. Army Corps of Engineers**
CECW-EH-Y **Washington, DC 20314-1000**

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# Engineering and Design
# RIVER HYDRAULICS

**1. Purpose.** This manual presents basic principles and technical procedures for analysis of open channel flows in natural river systems.

**2. Applicability.** This guidance applies to HQUSACE elements, major subordinate commands, laboratories, and field operating activities having civil works responsibilities.

**3. General.** Procedures described herein are considered appropriate and usable for planning, analysis and design of projects and features performed by the Corps of Engineers. Basic theory is presented as required to clarify appropriate application and selection of numerical models. This guidance also presents results of previous numerical model applications to river hydraulics and corresponding field observations.

FOR THE COMMANDER:

WILLIAM D. BROWN
Colonel, Corps of Engineers
Chief of Staff

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# Engineering and Design
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## Table of Contents


| Subject | Paragraph | Page |
|---|---|---|
| **Chapter 1** | | |
| **Introduction** | | |
| Purpose | 1-1 | 1-1 |
| Scope | 1-2 | 1-1 |
| Applicability | 1-3 | 1-1 |
| References | 1-4 | 1-1 |
| Needs for River Hydraulic Studies | 1-5 | 1-1 |
| General Methods | 1-6 | 1-1 |
| Organization | 1-7 | 1-2 |
| **Chapter 2** | | |
| **Introduction to River Hydraulics** | | |
| Introduction | 2-1 | 2-1 |
| Flow Dimensionality Considerations | 2-2 | 2-1 |
| Water Waves | 2-3 | 2-2 |
| Flow Classification | 2-4 | 2-4 |
| Regimes of Flow | 2-5 | 2-7 |
| Types of Flow | 2-6 | 2-7 |
| Classification of Flow Profiles | 2-7 | 2-9 |
| Basic Principles of River Hydraulics | 2-8 | 2-9 |
| **Chapter 3** | | |
| **Formulating Hydraulic Studies** | | |
| Initial Considerations | 3-1 | 3-1 |
| Overview of Techniques for Conducting River Hydraulics Studies | 3-2 | 3-1 |
| Analysis of Hydraulic Components | 3-3 | 3-4 |
| Data Requirements | 3-4 | 3-4 |
| Calibration of Hydraulic Analysis Models | 3-5 | 3-11 |
| Guidelines for Analytical Model Selection | 3-6 | 3-13 |
| **Chapter 4** | | |
| **Multidimensional Flow Analysis** | | |
| Introduction | 4-1 | 4-1 |
| Limitations of One-Dimensional Analysis | 4-2 | 4-1 |
| Equations of Flow | 4-3 | 4-1 |
| Significance of Terms | 4-4 | 4-2 |
| Use of Equations of Flow | 4-5 | 4-2 |
| Two-Dimensional Flow Conditions | 4-6 | 4-3 |
| Available Computer Programs | 4-7 | 4-4 |
| Data Requirements | 4-8 | 4-5 |
| Data Development and Model Calibration | 4-9 | 4-6 |
| Example Applications | 4-10 | 4-7 |
| **Chapter 5** | | |
| **Unsteady Flow** | | |
| Introduction | 5-1 | 5-1 |
| *Section I* | | |
| *Introduction* | | |
| Steady Versus Unsteady Flow Models | 5-2 | 5-1 |
| Conditions that Require Unsteady Flow Analysis | 5-3 | 5-2 |
| Geometry | 5-4 | 5-3 |
| Controls | 5-5 | 5-5 |
| Boundary Conditions | 5-6 | 5-5 |
| Steps to Follow in Modeling a River System | 5-7 | 5-9 |
| Accuracy of Observed Data | 5-8 | 5-11 |
| Calibration and Verification | 5-9 | 5-16 |
| Example Applications of Unsteady Flow Models | 5-10 | 5-17 |

| Subject | Paragraph | Page |
|---|---|---|
| *Section II* | | |
| *Theory of Routing Models* | | |
| Introduction | 5-11 | 5-17 |
| Unsteady Flow Model | 5-12 | 5-23 |
| Diffusion Model | 5-13 | 5-28 |
| Kinematic Wave Model | 5-14 | 5-29 |
| Accuracy of Approximate Hydraulic Models | 5-15 | 5-29 |
| Muskingum-Cunge Model | 5-16 | 5-30 |
| Hydrologic Routing Schemes | 5-17 | 5-30 |
| **Chapter 6** | | |
| **Steady Flow - Water Surface Profiles** | | |
| *Section I* | | |
| *Introduction* | | |
| Scope | 6-1 | 6-1 |
| Assumptions of the Method | 6-2 | 6-1 |
| Standard-step Solution | 6-3 | 6-1 |
| Range of Applicability | 6-4 | 6-2 |
| Example of Steady Flow Water Surface Profile Study | 6-5 | 6-3 |
| *Section II* | | |
| *Data Requirements* | | |
| Introduction to Data Requirements | 6-6 | 6-4 |
| Flow Regime | 6-7 | 6-5 |
| Starting Conditions | 6-8 | 6-5 |
| *Section III* | | |
| *Model Development* | | |
| Data Sources | 6-9 | 6-5 |
| Data and Profile Accuracy | 6-10 | 6-6 |
| Model Calibration and Verification | 6-11 | 6-7 |
| *Section IV* | | |
| *Special Problems* | | |
| Introduction to Special Problems | 6-12 | 6-13 |
| Bridge Hydraulics | 6-13 | 6-13 |
| Culvert Hydraulics | 6-14 | 6-16 |
| Limits of Effective Flow | 6-15 | 6-18 |
| Channel Controls | 6-16 | 6-19 |
| River Confluences | 6-17 | 6-19 |
| Changing Flow Regime | 6-18 | 6-19 |
| Ice-covered Streams | 6-19 | 6-23 |
| **Chapter 7** | | |
| **Water Surface Profiles with Movable Boundaries** | | |
| *Section I* | | |
| *Introduction* | | |
| Similarities and Differences Between Fixed and Mobile Bed Computations | 7-1 | 7-1 |
| *Section II* | | |
| *Theoretical Basis* | | |
| Sediment Transport Functions | 7-2 | 7-1 |
| *Section III* | | |
| *Data Requirements* | | |
| General Data Requirements | 7-3 | 7-4 |
| Geometric Data | 7-4 | 7-5 |
| Bed Sediment Data | 7-5 | 7-7 |
| Boundary Conditions Data | 7-6 | 7-7 |
| Data Sources | 7-7 | 7-11 |
| Data and Profile Accuracy | 7-8 | 7-14 |
| *Section IV* | | |
| *Model Confirmation and Utilization* | | |
| Model Performance | 7-9 | 7-14 |
| Development of Base Test and Analysis of Alternatives | 7-10 | 7-18 |
| *Section V* | | |
| *Computer Programs* | | |
| Introduction | 7-11 | 7-19 |
| Scour and Deposition in Rivers and Reservoirs (HEC-6) | 7-12 | 7-19 |
| Open Channel Flow and Sedimentation (TABS-2) | 7-13 | 7-20 |
| **Appendix A** | | |
| **References** | | |
| **Appendix B** | | |
| **Glossary** | | |

| Subject | Paragraph | Page |
|---|---|---|
| **Appendix C** | | |
| **Study Planning and Reporting** | | |
| Development of the Hydraulic Study Work Plan | C-1 | C-1 |
| Reporting Requirements | C-2 | C-2 |
| Hydrologic Engineering Study Checklist | C-3 | C-4 |
| Documentation Checklist | C-4 | C-4 |
| Example Detailed Hydrologic Engineering Management Plan for a Feasibility Study (Flood Damage Reduction using HEC-1 and -2) | C-5 | C-4 |
| Generic Hydraulic Study Work Plan for Unsteady, Gradually-Varied Flow Analysis (TABS-2) | C-6 | C-9 |
| **Appendix D** | | |
| **River Modeling - Lessons Learned** | | |
| *Section I* | | |
| *Defining River Geometry* | | |
| Introduction | D-1 | D-1 |
| Geometric Data | D-2 | D-1 |
| Developing Cross-Sectional Data to Define Flow Geometry | D-3 | D-3 |
| Developing Cross-Sectional Data to Satisfy Requirements of the Analytical Method | D-4 | D-6 |
| Reviewing Computed Results to Determine Adequacy of Cross-Sectional Data | D-5 | D-7 |
| Other Considerations in Developing Cross-Sectional Data | D-6 | D-7 |
| Modeling Flow Geometry at Structures | D-7 | D-7 |
| Developing Reach Length Data | D-8 | D-12 |
| Survey Methods for Obtaining Cross Sections and Reach Lengths | D-9 | D-14 |
| *Section II* | | |
| *Energy Loss Coefficients* | | |
| Variation of Manning's *n* with River Conditions | D-10 | D-15 |
| Estimation of *n* Values | D-11 | D-17 |
| Contraction and Expansion Losses | D-12 | D-22 |

# Chapter 1
# Introduction

## 1-1. Purpose

This manual presents the techniques and procedures that are used to investigate and resolve river engineering and analysis issues and the associated data requirements. It also provides guidance for the selection of appropriate methods to be used for planning and conducting the studies. Documented herein are past experiences that provide valuable information for detecting and avoiding problems in planning, performing, and reporting future studies. The resolution of river hydraulics issues always requires prediction of one or more flow parameters; be it stage (i.e., water surface elevation), velocity, or rate of sediment transport. This manual presents pragmatic methods for obtaining data and performing the necessary computations; it also provides guidance for determining the components of various types of studies.

## 1-2. Scope

Procedures for conducting river hydraulic investigations are presented herein with minimal theory. Details of the theoretical principles of river hydraulics can be found in standard textbooks and publications that are referenced throughout this manual. Each chapter provides general information and guidance to assist and support decisions regarding choice of the most appropriate analytical and/or modeling methods and data acquisition for specific circumstances.

## 1-3. Applicability

This guidance applies to HQUSACE elements, major subordinate commands, laboratories, and field operating activities having civil works responsibilities.

## 1-4. References

References are listed in Appendix A.

## 1-5. Needs for River Hydraulics Studies

Missions of the Corps of Engineers include the development and maintenance of flood control and navigation systems. It is the policy of the Corps of Engineers to plan, design, construct, and provide for the maintenance of safe, functional, cost-effective projects. River hydraulic analyses are an essential component of most riverine projects, and the results from these analyses are often critical to project formulation, design, construction, and operation throughout the project's life. River hydraulics includes the evaluation of flow characteristics and geomorphic (physical) behavior of rivers and changes in these due to natural or man-made conditions.

As examples, determination of the elevations of dams, spillways, levees, and floodwalls requires both hydrologic and hydraulic computations. A major component of studies related to floodplain information, flood control channel design, navigation, water quality assessment, environmental impact and enhancement analysis, is the prediction of stage, discharge, and velocity as functions of time anywhere on a river. Environmental aspects of river engineering often require the prediction of stage, velocity distributions, sediment transport rates, and water quality characteristics, to evaluate the impacts of proposed actions on future river characteristics. Study of any type of river project requires a thorough evaluation of the possible impacts that it may have, both upstream and downstream from the location of the project itself. Prediction of the operation, maintenance, and repair or replacement requirements of existing and proposed projects is another role that river hydraulics studies play in the Corps' planning and design processes.

## 1-6. General Methods

Reliable assessment and resolution of river hydraulics issues depend on the engineer's ability to understand and describe, in both written and mathematical forms, the physical processes that govern a river system. Provided herein are background information and technical procedures necessary to perform river hydraulics engineering studies. This manual provides river engineers at all levels of experience with a wide range of practical field examples, diagnostic advice, and guidance for performing river hydraulics investigations. Three categories of methods for predicting river hydraulic conditions were identified by Rouse (1959). The first and oldest uses engineering experience acquired from previous practice by an individual. The second utilizes laboratory scale models (physical models) to replicate river hydraulic situations at a specific site or for general types of structures. Laboratory modeling has been in extensive and successful use for at least the past 60 years. The third category is application of analytical (mathematical) procedures and numerical modeling. Recent use of physical and numerical modeling in combination, guided by engineering experience, is termed "hybrid modeling" and has been very successful.

*a. Field experience.* Field experience is an extremely valuable asset for an engineer, yet planning and design based only on experience may not yield a defensible and reproducible product. Design by experience alone may result in inefficient trial-and-error procedures. Furthermore, the rationale for the design may be lost if the person with the experience becomes unavailable.

*b. Physical models.* Application of physical models has evolved into a dependable and reproducible procedure for analyzing river hydraulics. Physical modeling techniques are documented by the U.S. Department of the Interior (1980), Petersen (1986), and ASCE (1942). These references provide guidance for planning and conducting river hydraulics studies using physical models.

*c. Analytical procedures.* Application of analytical (mathematical) procedures and numerical modeling have become accepted methods for analyzing river hydraulics and are the focus of this manual.

*d. River behavior.* The most thorough contemporary strategy for analyzing and predicting river behavior and response to imposed changes combines all three of the methods mentioned above; this is known as hybrid modeling.

## 1-7. Organization

Seven chapters, followed by four appendixes, detailing guidelines, data requirements, and computational procedures are presented. The chapters are: Introduction, Introduction to River Hydraulics, Formulating Hydraulic Studies, Multidimensional Flow Analysis, Unsteady Flow, Steady Flow - Water Surface Profiles, and Water Surface Profiles With Movable Boundaries. Guidance for selecting appropriate study and design procedures is given in each chapter along with examples. The order of the technical chapters (4, 5, 6, and 7) is intended to show how each successive approach derives from the prior approach. References are in Appendix A. Appendix B provides definitions of the technical terms used throughout this document. Appendix C overviews reporting requirements and the development of a study work plan. Appendix D gives guidance on the preparation of geometric data and selection of energy loss coefficients based upon past experience. This information is generally applicable to all the methods presented in this manual; therefore, Appendix D should be consulted prior to embarking on any river hydraulics study. This manual is not intended to be read straight through; there is, therefore, some redundancy among Chapters 4, 5, 6, 7, and Appendix D with regard to such items as calibration procedures and parameter selection.

# Chapter 2
# Introduction to River Hydraulics

## 2-1. Introduction

Proper use of this manual requires knowledge of the fundamentals and laws of fluid mechanics. This chapter provides an overview of the principles necessary to perform river hydraulic studies and provides some guidance for selecting appropriate methods for conducting those studies. It must be supplemented with use of standard textbooks such as Chow (1959), Henderson (1966), and/or French (1985). Topics presented herein include: flow dimensionality, the nature of water and flood waves, an overview of definitions and flow classifications, and basic principles of river hydraulics and geomorphology.

*a. General.* Rivers are complex and dynamic. It is often said that a river adjusts its roughness, velocity, slope, depth, width, and planform in response to human activities and (perhaps associated) changing climatic, geologic, and hydrologic regimes. These adjustments may be rapid or slow, depending upon the source and character of the forces spawning the adjustments. When a river channel is modified locally, that modification may initiate changes in the channel and flow characteristics that may propagate both upstream and downstream and throughout tributary systems. These changes may occur over large distances and persist for long times.

*b. Analysis techniques.* Effective analysis of river problems requires recognition and understanding of the governing processes in the river system. There are two basic items that must always be considered in river hydraulics analyses: the characteristics of the flow in the river, and the geomorphic behavior of the river channel. These two components are sometimes treated separately; however, in alluvial channels (channels with movable boundaries) the flow and the shape of the boundary are interrelated. One-dimensional, steady state, fixed-bed water surface profiles are often computed as part of "traditional" river hydraulics studies. However, some floodplain management, flood control, or navigation studies may require consideration of unsteady (time-dependent) flow, mobile boundaries (boundary characteristics that can change with flow and time), or multi-dimensional flow characteristics (flows with nonuniform velocity distributions) to properly perform the required studies.

*c. Options.* The analyst has a number of options for analyzing river flows and must choose one (or a combination of several) that yields sufficiently useful and defensible results at optimal cost. There does not yet exist definitive criteria which can be routinely applied to yield a clear choice of method. This manual serves as a guide for thought processes to be used by the hydraulic engineer studying a reach of river with the aim of predicting its behavior for a wide range of flows.

## 2-2. Flow Dimensionality Considerations

*a. Realm of one-dimensionality.* To decide if a multidimensional study is needed, or a one-dimensional approach is sufficient, a number of questions must be answered. Is there a specific interest in the variation of some quantity in more than one of the possible directions? If only one principal direction can be identified, there is a good possibility that a one-dimensional study will suffice. Let this direction be called the main axis of the flow (e.g., streamwise); it is understood that that direction can change (in global coordinates) along the flow axis, as in a natural river.

*b. Limitations of one dimensionality.* One-dimensional analysis implies that the variation of relevant quantities in directions perpendicular to the main axis is either assumed or neglected, not computed. Common assumptions are the hydrostatic pressure distribution, well-mixed fluid properties in the vertical, uniform velocity distribution in a cross section, zero velocity components transverse to the main axis, and so on.

*c. Two-dimensional flow.* It is possible that actual transverse variations will differ so greatly from the assumed variation that streamwise values, determined from a one-dimensional study, will be in significant error. If flow velocities in floodplains are much less than that in the main channel, actual depths everywhere will be greater than those computed on the basis of uniform velocity distribution in the entire cross section. It is possible that the transverse variations will be of greater importance than the streamwise values. This is of particular importance when maximum values of water surface elevation or current velocity are sought. For example, in river bends, high velocities at one bank can lead to scour that would not be predicted on the basis of average streamwise values. Also, flow in a bend causes super-elevation of the water surface on the outside of the bend which may be a significant source of flooding from a dam-break wave passing through a steep alpine valley.

In swiftly flowing streams, the superelevation of the water surface on the outside of a bend, required to accelerate the water towards the inside in making the turn, needs not disrupt the one-dimensionality of the flow from the computational standpoint. The superelevation is predictable from the one-dimensional computed velocity and the bend radius, and can be added to the water surface elevation at the stream axis after this has been computed. For a third example, a strong cross wind in a wide shallow estuary can generate water surface elevations considerably greater on the downwind bank than on the main axis of the channel.

*e. Determination of flow dimensionality.* It is not possible to state with theoretical certainty that a given reach can be assumed one-dimensional unless multi-dimensional studies on the reach have been carried out and compared to the results of a one-dimensional approach. As a practical rule of thumb, however, if the reach length is more than twenty times the reach width, and if transverse flow and stage variations are not specifically of interest, the assumption of one dimensionality will likely prove adequate. Events of record in wide reaches can yield indications of susceptibility to strong cross winds or large transverse differences in atmospheric pressures. The history of flooding in the reach should be studied for potential sources of significant transverse disturbance. As an extreme example, it was the massive failure of the left bank, which fell into the reservoir, that produced the catastrophic overtopping of Viaont Dam in Italy in 1963, and it was the ride up of the resulting wave from the dammed tributary which crossed the channel of the main stream, the Piave River, and obliterated the town of Longarone. In most cases departures from strictly one-dimensional flow are confined to regions in the vicinity of local disturbances. Expansions and contractions in cross sections lead to transverse nonuniform velocity distributions and, if severe enough, in water surface elevations as well. These local effects are usually accounted for in a one-dimensional analysis by adjusting coefficients for head loss.

*f. Composite channels.* The concept of a composite channel is typically used to account for retardation of flow by very rough floodplains in a one-dimensional analysis. It is assumed that, with a horizontal water surface and energy slope common to main channel and overbank flows, the total discharge can be distributed among the main channel and overbanks in proportion to their individual conveyances. The different length traveled by the portion of the flow in the floodplains can, in principle, be accommodated by computing three contiguous one-dimensional flows, the main channel, and the right and left floodplains (Smith 1978, U.S. Army Corps of Engineers 1990b).

*g. Floodplains.* A river rising rapidly and going overbank may take significant time to inundate the floodplain. The transverse water surface will then not be horizontal and will slope downward (laterally outward from the main channel) to provide the force for the flood proceeding up the floodplain. The cross-sectional area for carrying the streamwise flow will then be less than that under a horizontal line at the elevation of the water surface in the main channel. In the absence of two-dimensional computations, information from past records of the timing of floodplain inundation should be compared to rise time in the main channel to determine the importance of this effect.

*h. Networks.* While a network of interconnected streams is surely two-dimensional, the individual channels comprising each reach of the network can usually be treated as one-dimensional. In some cases of multiple flow paths, such as through bridges crossing wide floodplains with multiple asymmetric openings, the flow distribution may be difficult to determine and the water surface elevation substantially non-horizontal; in such cases, two-dimensional modeling may be preferable (U.S. Department of Transportation 1989).

## 2-3. Water Waves

*a. General.* Water flowing (or standing) with a free surface open to the atmosphere is always susceptible to wave motion. The essence of wave motion exists in the concept of the propagation of disturbances. If a given flow is perturbed by something somewhere within its boundaries, some manifestation of that perturbation is transmitted at some velocity of propagation to other portions of the water body. There are different categories of water waves, many of which are not pertinent to river hydraulics studies. A pebble cast into a body of water generates waves which radiate from the point of entry in all directions at speeds, relative to the bank, dependent upon the water velocity and depth. In still water they radiate as concentric circles. The concept of wave propagation depending upon wave celerity and water velocity is common to the analysis of all water waves. The waves generated by a dropped pebble are usually capillary waves, whose celerity is strongly dependent upon the surface tension at the air-water interface. They are unrelated to river hydraulics except that they may affect measurements in a small-scale physical model of a channel.

*b. Wave types.*

(1) Chop and swell on the surface of an estuary in a stiff wind represent gravity waves, which are unlike a flood wave in a river because the motions of the water particles are confined to orbits in the upper layers of the water body. The deeper a measurement is taken below the surface of such a wave, the smaller are the velocities. The celerities of such waves depend mainly upon the size of the wave, and less upon the depth of the water upon whose surface they travel. Such waves can cause substantial intermittent wetting, erosion, and even ponding well above the surface of an otherwise undisturbed water body. Their short wavelength implies variation of velocities and pressures in the vertical as well as in the horizontal directions with time; hence, the mathematics of their calculation is substantially more complicated than that of flood waves. In typical flood studies, the magnitudes of such surface waves are estimated from empirical formulas and then superimposed upon the surface of the primary flood wave. Another kind of short wave occurring in very steep channels at Froude numbers (see paragraph 2-4*c*) near two results from the instability of flow on those slopes. This form of wave motion is the so-called "roll wave," and can be seen in steep channels, such as spillways with small discharges (e.g., gate leakage).

(2) There is another variety of short wave that may be pertinent to some flood waves. In rare instances, changes in flow are so extreme and rapid that a hydraulic bore is generated. This is a short zone of flow having the appearance of a traveling hydraulic jump. Such a jump can travel upstream (example: the tidal bore when the tide rises rapidly in an estuary), downstream (example: the wave emanating from behind a ruptured dam), or stay essentially in one place (example: the hydraulic jump in a stilling basin).

*c. Flood waves.* The essence of flood prediction is the forecasting of maximum stages in bodies of water subject to phenomena such as precipitation runoff, tidal influences (including those from storm tides), dam operations, and possible dam failures. Also of interest are discharge and stage hydrographs, velocities of anticipated currents, and duration of flooding. Deterministic methods for making such predictions, typically called flood routing, relate the response of the water to a particular flow sequence. A brief introduction is given here; details and examples are in Chapter 5 and Appendix D. Only one-dimensional situations are discussed here; that is, river reaches in which the length is much greater than the width. Similarly, it is assumed that the boundaries of the reach are rigid and do not deform as a result of the flow (see Chapter 7 and EM 1110-2-4000, 1989).

(1) Flood routing. Many flood routing techniques were developed in the late nineteenth and early twentieth centuries. The fact that water levels during flood events vary with both location and time makes the mathematics for predicting them quite complicated. Various simplifying assumptions were introduced to permit solutions with a reasonable amount of computational effort. While analytical techniques for solving linear wave equations were known, those solutions could not, in general, be applied to real floods in real bodies of water because of the nonlinearity of the governing equations and the complexity of the boundaries and boundary conditions. Numerical solutions of the governing equations were largely precluded by the enormous amount of arithmetic computation required. The advent and proliferation of high-speed electronic computers in the second half of the twentieth century revolutionized the computation of flood flows and their impacts. Numerical solutions of the governing partial differential equations can now be accomplished with reasonable effort.

(2) Data for flood routing. Solution of the partial differential equations of river flow requires prescription of boundary and initial conditions. In particular, the geometry of the watercourse and its roughness must be known, as well as the hydraulic conditions at the upstream and downstream ends of the reach and at all lateral inflows and outflows (tributaries, diversions) along the reach. Due to the extreme irregularity of a natural watercourse, the channel geometry and hydraulic properties (such as roughness and infiltration) cannot be specified exactly. The accuracy to which they must be specified to yield reliable results is not a trivial issue (U.S. Army Corps of Engineers 1986, 1989).

(3) Water motion. The motion of water particles at a cross section during a flood is nearly uniform, top to bottom. The drag of the sides and bottom, possible secondary currents resulting from channel bends or irregularities, and off-channel storage (ineffective flow) areas create a nonuniform distribution of velocity across a cross section. The celerity of a flood wave is dependent, in a fundamental way, on the water depth. In a flood wave, the pressure distribution is nearly hydrostatic; i.e., it increases uniformly with depth below the surface.

These are so-called "long waves" that are, in fact, gradually varied unsteady flows in open channels. The term "unsteady" implies that measurements of water velocity at one point in such a channel will show time variance at a scale larger than turbulent fluctuations. "Varied" means that, at any instant, velocities at different points along the channel are different. "Gradually varied" means that the pressure distribution in a cross section is hydrostatic.

(4) Wave speed. The analyst must be cognizant of the fact that the response of water in a river to a flood or other disturbance is a wave which propagates at some speed and influences water levels consecutively, not simultaneously. While it may be possible to ignore that fact under certain circumstances, it should never be done mechanically without careful consideration of the specific conditions. Only if the travel time of the wave is small compared to the time for a boundary condition to change substantially can the water in a reach be assumed to behave as a unit without regard for the wave motion. The kinematic wave speed, that is, the speed of propagation of the main body of the flood, is strongly dependent on the channel slope and roughness and must be considered (Ponce 1989).

## 2-4. Flow Classification

To determine which principles apply to a particular situation in river mechanics, it is necessary to properly classify the flow. Various categories of flow are amenable to different simplifying assumptions, data requirements, and methods of analysis. The first step in the analysis of river hydraulics situations is classification of the state, type, and characteristics of the flow. Once the presumed flow characteristics have been categorized, the engineer can identify the data, boundary conditions, and simulation techniques appropriate for the situation. The following sections present definitions and flow classifications that lead to selection of analysis techniques.

*a. Effects of channel boundaries.* Water may be conveyed in two types of conduits: (1) open channels and (2) pressure conduits (neglecting ground water). The extent to which boundary geometry confines the flow is an important basis for classifying hydraulic problems. Open channel flow is characterized by a free (open to atmospheric pressure) water surface. Pipe or pressure flow occurs in conduits, pipes, and culverts that are flowing completely full and, therefore, have no free water surface. Flow in a closed conduit, however, is not necessarily pipe or pressure flow. If it is flowing partially full and has a free surface, it must be classified and analyzed as open channel flow.

(1) Figure 2-1 shows that the same energy principles are valid for both pressure flow and open channel flow. The dynamic forces, however, in steady pressure flows are the viscous and inertial forces. In open channel flow the force of gravity must also be considered. Flows are more complicated in open channels because the water surface is free to change with time and space; consequently, the water surface elevation, discharge, velocity, and slopes of the channel bottom and banks are all interrelated. Also, the physical conditions (roughness and shape) of open channels vary much more widely (in space and time) than those of pipes, which usually have a constant shape and roughness. Because this manual covers only river hydraulics, little emphasis is placed on methods of solving pipe or pressure flow problems unless they pertain directly to river hydraulics, such as pressure flow through bridge crossings or culverts (see Chapter 6). Chow (1959, chap. 1) discusses many of the similarities and differences between pipe and open channel flow.

(2) Flow in an alluvial channel (a channel with movable boundaries) behaves differently from flow in a rigid boundary channel. In alluvial channels (most natural rivers) rigid boundary relationships apply only if the movement of the bed and banks is negligible during the time period of interest. Once general mobilization of bed and bank materials occurs, the flow characteristics, behavior, and shape of the channel boundaries become interrelated, thus requiring far more complex methods for flow analysis. Chapters 4, 5, and 6 of this manual are directed primarily at rigid boundary problems. Chapter 7 presents the theory and methods for analyzing movable boundary river hydraulics. Details of sediment investigations are provided in EM 1110-2-4000.

*b. Effects of viscosity (laminar and turbulent flow).*

(1) The behavior of flow in rivers and open channels is governed primarily by the combined effects of gravity and fluid viscosity relative to inertial forces. Effects of surface tension are usually negligible for natural rivers. The three primary states of flow are laminar flow, transitional flow, and turbulent flow.

(2) A flow is laminar, transitional, or fully turbulent depending on the ratio of viscous to inertial forces as defined by the Reynolds number:

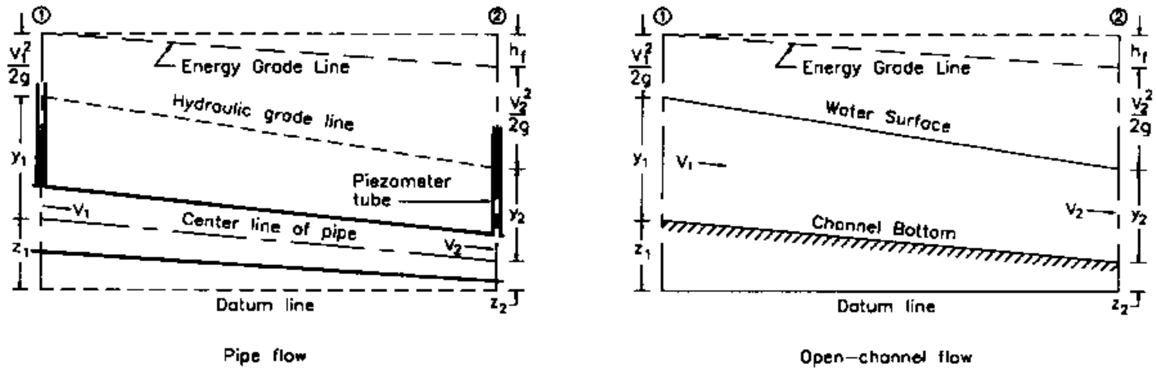


**Figure 2-1. Comparison between pipe flow and open-channel flow**

$$R_e = \frac{VL}{\nu} \tag{2-1}$$

where

$R_e$ = Reynolds number (dimensionless)
$V$ = characteristic flow velocity (ft/sec)
$L$ = characteristic length (ft)
$\nu$ = kinematic viscosity of water ($ft^2$/sec)

In open channels, $L$ is usually taken as the hydraulic radius; i.e., the cross-sectional area normal to the flow divided by the wetted perimeter. Care must be taken to use a homogeneous system of units for these terms so that the Reynolds number is dimensionless. An open channel flow is laminar if the Reynolds number is less than 500. Flows in open channels are classified as turbulent if the Reynolds number exceeds 2,000, and they are transitional if $R_e$ is between 500 and 2,000 (Chow 1959). Laminar flow is characterized by the dominant effects of viscosity. In laminar flow, parcels of fluid appear to travel in smooth parallel paths. Laminar flow occurs very rarely in natural open channels. When the surface of a river appears smooth or glassy, it does not necessarily mean that the flow is laminar; rather, it is most likely tranquil, though turbulent flow. Laminar open channel flow can occur, however, when a very thin sheet of water flows over a smooth surface; otherwise, it is usually restricted to specially controlled laboratory facilities.

(3) In turbulent flow, pulsatory cross-current velocity fluctuations cause individual parcels of fluid to move in irregular patterns, while the overall flow moves downstream. One effect of the microstructure of turbulent flow is the formation of a more uniform velocity distribution. Figure 2-2 shows the differences between typical laminar and turbulent velocity profiles in an open channel and a pipe. Much greater energy losses occur in turbulent flow. The energy required to generate the random cross current velocities must come from the total energy of the river, but it is of no real help in transporting the flow downstream. Therefore, open channel flow relations for turbulent flows describe energy and friction losses differently than for laminar flows.

(4) Because flows in natural rivers are always turbulent, methods of analyzing turbulent open channel flows are presented exclusively in this document. Readers interested in the analyses of laminar flow conditions

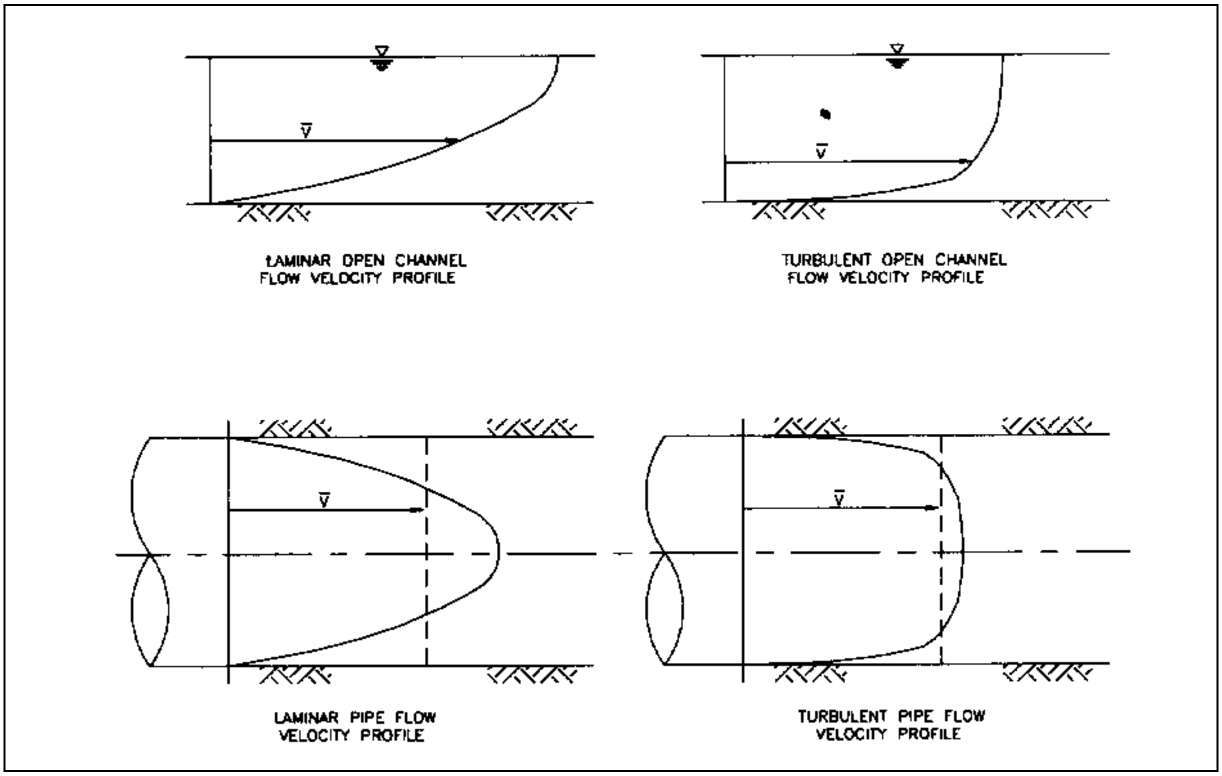


**Figure 2-2. Laminar and turbulent velocity profiles**

should refer to texts by Chow (1959), Henderson (1966), and Rouse (1959).

*c. Effects of gravity (subcritical and supercritical flow).* The ratio of inertial to gravitational forces is an important measure of the state of open channel flow and is represented by the Froude number:

$$F = \frac{V}{\sqrt{gL}} \qquad (2\text{-}2)$$

where

$F$ = Froude number (dimensionless)
$V$ = mean flow velocity in the channel (ft/sec)
$g$ = acceleration of gravity (ft/sec$^2$)
$L$ = characteristic length term (ft)

In open channels and rivers the characteristic length ($L$) is often taken as the hydraulic depth; i.e., the cross-sectional area normal to the flow divided by the top width at the free surface. Depending on the magnitude of the Froude number, the state of flow is either "sub-critical', "critical", or "supercritical."

(1) When the Froude number is less than 1, the effects of gravitational forces are greater than inertial forces, and the state of the flow is referred to as subcritical, or tranquil flow. Note that the denominator in the Froude number (Equation 2-2) is the expression for celerity of a shallow water wave. Therefore, in subcritical flow, the wave celerity is greater than mean channel velocity, and a shallow water wave can move upstream. As a simple field test, toss a stone into the river; if you observe the ripples from the stone hitting the water moving upstream, the flow for that location, depth, and discharge is subcritical ($F < 1$).

(2) When inertial and gravitational forces are equal, the Froude number is equal to unity, and the flow is said to be at the critical state (i.e., critical flow). For these conditions, a shallow water wave remains approximately stationary in the flow relative to the banks. At critical flow, the depth is referred to as "critical depth."

(3) When inertial forces exceed gravitational forces ($F > 1$) the state of flow is referred to as supercritical, or rapid flow. For this state, the flow is characterized by high velocity, and shallow water waves are immediately

carried downstream. It is possible, however, that point velocities in a natural channel will exceed critical velocity when the average state of flow is subcritical.

(4) Prior to performing hydraulic calculations, such as determining water surface profiles, engineers must determine the state of flow for the range of discharges and depths being evaluated. When the state of flow is subcritical (F < 1), the water surface profile is controlled by channel characteristics at the downstream end of the river reach. Therefore, steady flow water surface profile computations proceed from the downstream control point upstream (referred to as a backwater calculation). If supercritical flow exists, calculations go from upstream to downstream. If the direction of the computation does not correspond to the prevailing state of flow, the computed water surface profile can diverge from the true profile and lead to erroneous results. If computations proceed in the proper direction for the state of flow, the calculated water surface profile converges to the true profile even if the estimated starting water surface is in error.

## 2-5. Regimes of Flow

There are four regimes of open channel flow, depending on the combined effects of viscosity and gravity: (1) subcritical-laminar, (2) subcritical-turbulent, (3) supercritical-laminar, and (4) supercritical-turbulent. The two laminar regimes are not relevant to natural rivers because fully turbulent flow is always the case. Therefore, determination of the flow regime for most open channel and river hydraulics situations involves verifying that the state of the flow is either subcritical (F < 1) or supercritical (F > 1).

*a. Subcritical flow.* In rivers and channels, if the flow is subcritical (F < 1) and the bed immobile, water will accelerate over shallow humps and obstructions on the bottom and decelerate over deeper areas and troughs. This is illustrated in Figure 2-3. In sand bed channels flow separation often occurs just downstream of the crest of the sand waves. Surface boils may appear on the water surface just downstream from the flow separation locations. In natural alluvial channels, the occurrence of separation zones and increased flow turbulence leads to increases in flow resistance and energy losses.

*b. Supercritical flow.* If the flow is supercritical (F > 1), water flowing over obstructions and humps will decelerate while accelerating in the pools and troughs as shown in Figure 2-3.(c) and (d), respectively. The interaction and effects of the flow with a mobile alluvial bed are presented in Chapter 7.

## 2-6. Types of Flow

The following flow classifications are based on how the flow velocity varies with respect to space and time. Figure 2-4 shows some of the possible types of open channel flow that occur in rivers. Each type of flow must be analyzed using methods that are appropriate for that flow.

*a. Steady flow.* A flow is steady if the velocity at a specific location does not change in magnitude or direction with time. (Turbulent fluctuations are neglected in these definitions.)

*b. Unsteady flow.* If the velocity at a point changes with time, the flow is unsteady. Methods for analyzing unsteady flow problems account for time explicitly as a variable, while steady flow methods neglect time all together.

*c. Uniform flow.* Uniform flow rarely occurs in natural rivers because, by definition, uniform flow implies that the depth, water area, velocity, and discharge do not change with distance along the channel. This also implies that the energy grade line, water surface, and channel bottom are all parallel for uniform flow. The depth associated with uniform flow is termed "normal depth." Uniform flow is considered to be steady flow only, since unsteady uniform flow is practically nonexistent (Chow 1959). Only in a long reach of prismatic channel of uniform roughness carrying a flow that has been undisturbed at the reach boundaries for a long time will the flow be uniform.

*d. Nonuniform flow.* Most flow in natural rivers and channels is nonuniform, or spatially varied flow. Here, the term "spatially varied" is to be taken in the one-dimensional sense; i.e. hydraulic variables vary only along the length of the river. Even if the flow is steady, spatial variation can result from changes occurring along the channel boundaries (e.g., channel geometry changes), from lateral inflows to the channel, or both.

(1) Rapidly varied. If spatial changes to the flow (depth and/or velocity) occur abruptly and the pressure distribution is not hydrostatic, the flow is classified as rapidly varied. Rapidly varied flow is usually a local

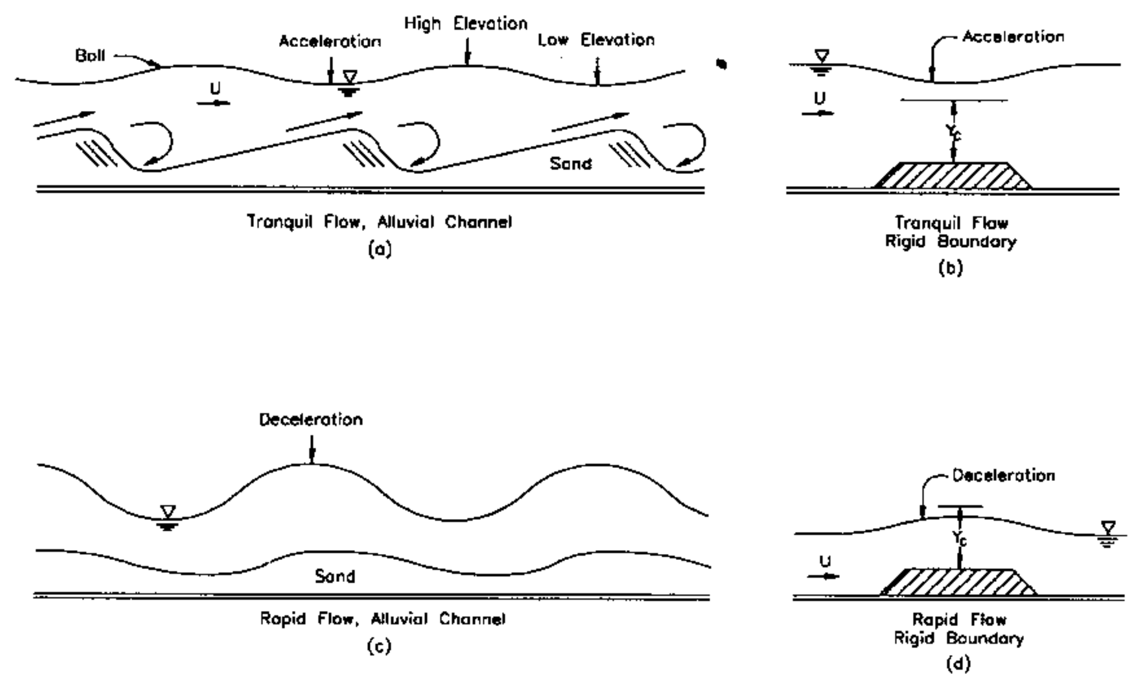


**Figure 2-3. Relation between water surface and bed configuration for tranquil and rapid flow (from Simons and Sentürk 1976)**

phenomenon. Examples are the hydraulic jump and hydraulic drop (see p. 6 of Chow 1959).

(2) Gradually varied. As a rule of thumb, if the slope of the surface of a body of water is indiscernible to the naked eye, the flow therein is gradually varied. Unsteadiness of open channel flow (in contrast to the case of a rigid closed conduit flowing full) implies non-uniformity because disturbances (imposed flow changes) are always propagated as waves. In principle, at any instant, some portion of the flow is influenced by the disturbance, other portions have not yet been reached, and the requirements for varied, i.e., nonuniform flow are met. Furthermore, any nonuniformity of the channel characteristics; e.g., expansions and contractions in cross section shape or changes in slope or roughness, causes the flow to accelerate and decelerate in response. The relative sizes of these two contributions to the flow non-uniformity, flow unsteadiness, and irregular channel geometry, influence the applicability of various techniques for simulating river flows. In general, the flow in a river subject to variations in inflow, outflow, or tidal action should be assumed to be unsteady and non-uniform. Gradually varied flow implies that the stream lines are practically parallel (e.g., a hydrostatic pressure distribution exists throughout the channel section). An underlying assumption for gradually varied flow computations is that "*The headloss for a specified reach is equal to the headloss in the reach for a uniform flow having the same hydraulic radius and average velocity* ..." (French 1985, p. 196). This assumption allows uniform flow equations to be used to model the energy slope of a gradually varied flow at a given channel section. It also allows the coefficient of roughness (Manning's *n*), developed for uniform flow, to be applied to varied flows. These assumptions have never been precisely confirmed by either experiment or theory, but the errors resulting from them are known to be small compared to other errors such as survey errors and roughness estimation (U.S. Army Corps of Engineers 1986). If large errors are introduced by the use of simplified gradually varied flow methods, or if the particular flow conditions violate the basic assumptions of steadiness, one-dimensionality, or rigid boundaries, the river engineer must consider use of more detailed analytical methods. Chapter 3 presents some simple procedures for eliminating inappropriate methods and identifying

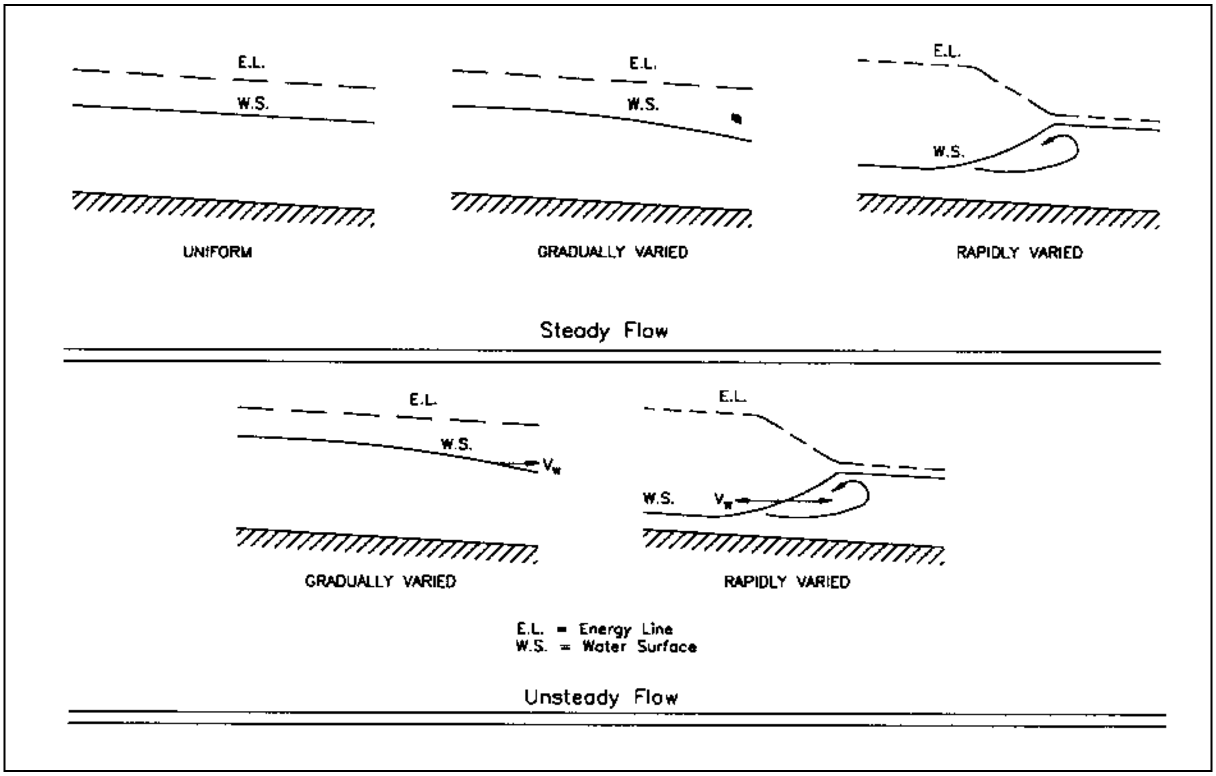


**Figure 2-4. Some types of open-channel flow**

what methods may be appropriate for any particular study.

## 2-7. Classification of Flow Profiles

The following classification of steady flow water surface profiles follows that of Chow (1959). This assumes a one-dimensional condition.

*a. Channel slope.* Channel slope is one criterion used to classify steady flow profiles. A critical slope is one on which critical velocity is sustained by a change in potential energy rather than pressure head. A mild slope is less than critical slope, and a steep slope is greater than critical slope for a given flow. When the slope is positive, it is classified as mild, steep, or critical, and the corresponding flow profiles are the M, S, or C profiles, respectively (see Figure 2-5). If the slope of the channel bed is zero, the slope is horizontal and the profiles are called H profiles. If the bed rises in a downstream direction, the slope is negative and is called an adverse slope, producing A profiles.

*b. Normal and critical depths.* Another parameter used to classify gradually varied flow profiles is the magnitude of the water depth relative to normal depth, $D_n$, and critical depth, $D_c$. The depth that would exist if the flow were uniform is called normal depth. Critical depth is that for which the specific energy for a given discharge is at a minimum. Specific energy is defined as:

$$H_e = d + \frac{\alpha V^2}{2g} \tag{2-3}$$

where

$d$ = depth of flow (ft)
$\alpha$ = energy correction factor (dimensionless)
$V^2/2g$ = velocity head (ft)

## 2-8. Basic Principles of River Hydraulics

*a. Conservation of mass.* Evaluation of the hydraulic characteristics of rivers and open channels requires

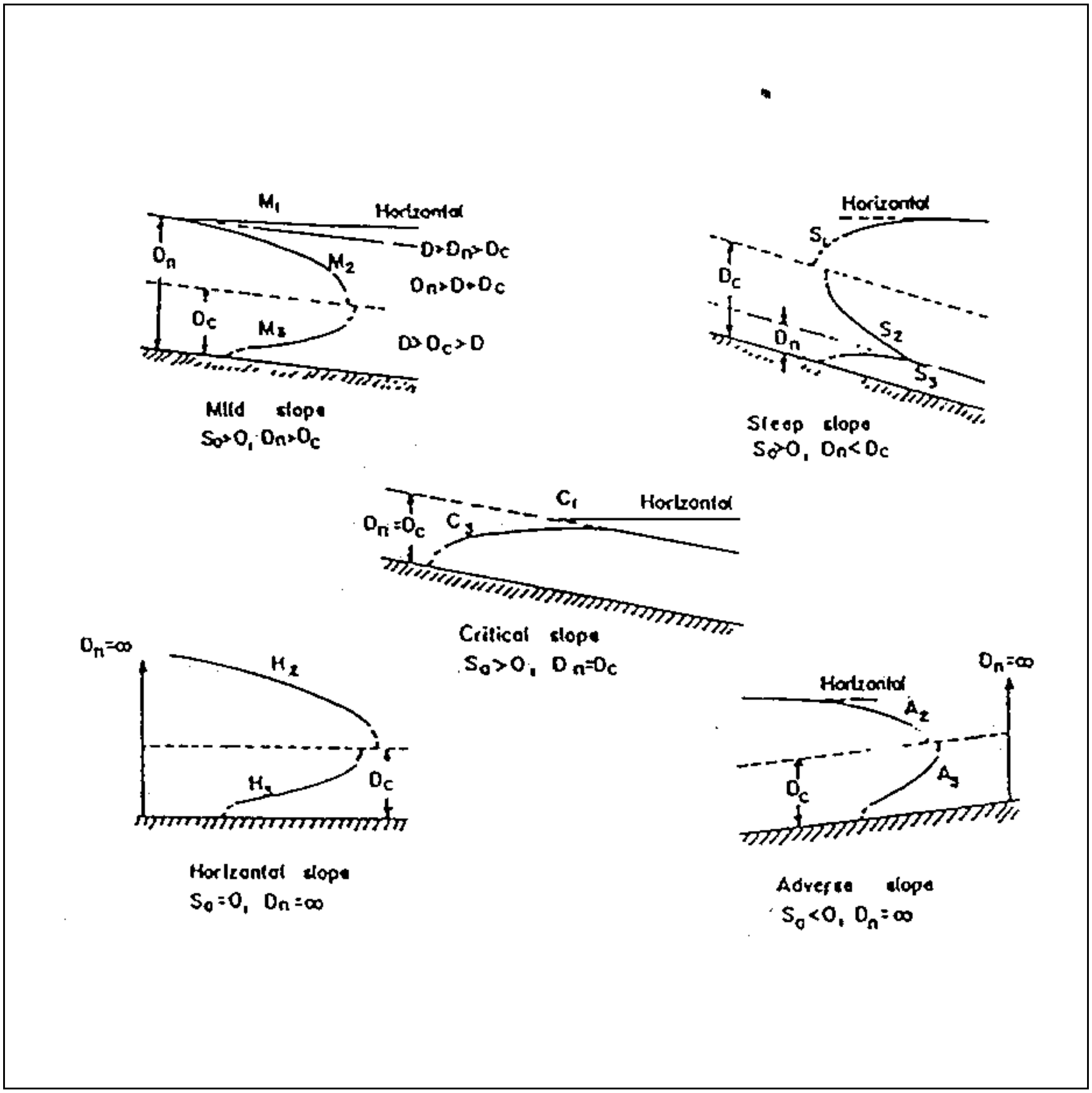


**Figure 2-5. Classification of steady flow profiles**

analysis of mass and energy conservation. Conservation of mass is often referred to as flow continuity. Continuity is the principle that states that mass (stream flow volume) is conserved (e.g., mass is neither created nor destroyed within the system being evaluated). Mass conservation in a volumetric sense means that the volume passing a given location will also pass another location downstream provided that changes in storage, tributary inflows and outflows, evaporation, etc. between the two locations are properly accounted for.

(1) The simplest description of mass conservation for steady, one-dimensional, flow without intervening inflows and outflows is:

$$Q = V_1 \times A_1 = V_2 \times A_2 = \ldots V_i \times A_i \qquad (2\text{-}4)$$

where

$Q$ = volumetric flow rate ($ft^3$/sec)
$V$ = mean flow velocity (ft/sec)
$A$ = cross-sectional flow area ($ft^2$)

and the subscripts on V and A designate different river section locations. Equation 2-4 is not valid where the discharge changes along the river. That type of flow is referred to as spatially varied flow and occurs when water runs into or out of the river from tributaries, storm drains, drainage canals, and side-channel spillways.

(2) The continuity equation for unsteady, one-dimensional flow requires consideration of storage as shown below:

$$B \frac{\alpha d}{\alpha t} + \frac{\alpha Q}{\alpha x} = 0 \tag{2-5}$$

where

$B$ = channel top width (ft)
$x$ = longitudinal distance along the centerline of the channel (ft)
$d$ = depth of flow (ft)
$t$ = time (seconds)

The two terms represent the effects of temporal change in storage and spatial change in discharge, respectively. Further detail regarding the derivation and alternative forms of the continuity equation are presented by Chow (1959), Henderson (1966), and French (1985). See also Chapters 4 and 5.

*b. Conservation of energy.* The second basic component that must be accounted for in one-dimensional steady flow situations is the conservation of energy. The mathematical statement of energy conservation for steady open channel flow is the modified Bernoulli energy equation; it states that the sum of the kinetic energy (due to motion) plus the potential energy (due to height) at a particular location is equal to the sum of the kinetic and potential energies at any other location plus or minus energy losses or gains between those locations. Equation 2-6 and Figure 2-6 illustrate the conservation of energy principle for steady open channel flow.

$$WS_2 + \frac{\alpha_2 V_2^2}{2g} = WS_1 + \frac{\alpha_1 V_1^2}{2g} + h_e \tag{2-6}$$

where

$WS$ = water surface elevation (ft)
$h_e$ = energy loss (ft) between adjacent sections

and the other terms were previously defined. This equation applies to uniform or gradually varied flow in channels with bed slopes (θ) less than approximately 10 degrees. Units of measurement are cited in Table 2-1. In steeper channels, the flow depth 'd' must be replaced with (d*cosθ) to properly account for the potential energy. For unsteady flows refer to Chapters 4 and 5.

**Table 2-1**
**Conversion Factors, Non-SI to SI (Metric) Units of Measurement**
**Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:**

| Multiply | By | To Obtain |
|---|---|---|
| cubic feet | 0.02831685 | cubic meters |
| cubic yards | 0.7645549 | cubic meters |
| degrees Fahrenheit | 5/9* | degrees Celsius or Kelvin |
| feet | 0.3048 | meters |
| inches | 2.54 | centimeters |
| miles (US statute) | 1.609347 | kilometers |
| tons (2,000 pounds, mass) | 907.1847 | kilograms |

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: C = (5/9)(F - 32). To obtain Kelvin (K) readings, use: K = (5/9)(F - 32) + 273.15.

*c. Application to open channels.* Even though the same laws of conservation of mass and energy apply to pipe and open channel flow, open channel flows are considerably more difficult to evaluate. This is because the location of the water surface is free to move temporally and spatially and because depth, discharge, and the slopes of the channel bottom and free surface are interdependent (refer to Figure 2-1 and to Chow (1959) for further explanation of these differences). In an open channel, if an obstruction is placed in the flow and it generates an energy loss ($h_e$ in Figure 2-6), there is some distance upstream where this energy loss is no longer reflected in the position of the energy grade line, and thus the flow depth at that distance is unaffected. The flow conditions will adjust to the local increase in energy loss by an increase in water level upstream from the disturbance thereby decreasing frictional energy losses. This allows the flow to gain the energy required to

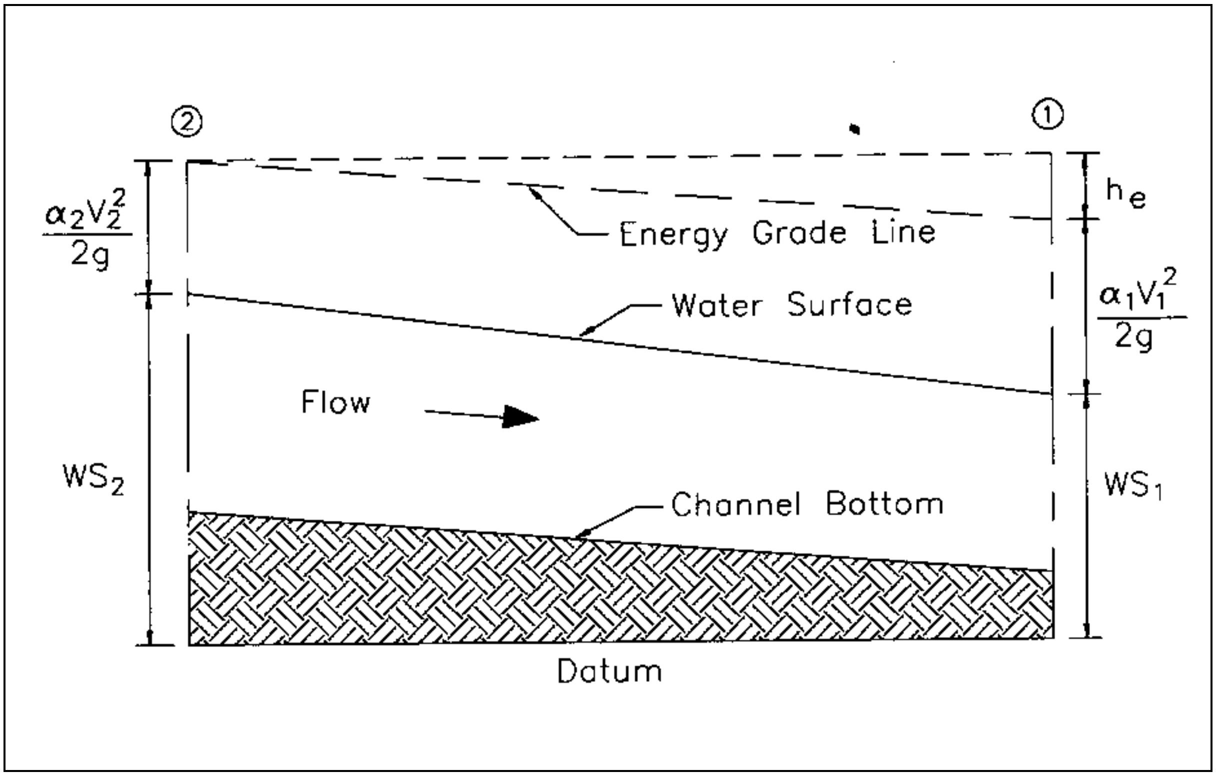


**Figure 2-6. Open channel energy relationships**

overcome the local energy loss, but the increase will gradually decrease in the upstream direction. It is this complication, the freedom in the location of the water surface, that makes hydraulics of open channels more complicated and difficult to evaluate than that of closed conduits.

*d. Use in natural rivers.* The primary difference between study methods used for prismatic channels (channels with an unvarying cross section, roughness, and bottom slope) and natural rivers results from variations in natural river channel cross-sectional shape and roughness and variable bottom slope. Figure 2-7 presents plan and profile views of a typical study reach for a natural river and identifies the various classes and types of flow that may occur within the reach. Note that, not only can the type of flow vary along a natural channel, but also the flow regime. Practical application of steady, one-dimensional flow theory is detailed in Chapter 6.

(1) Figure 2-7 emphasizes that, in natural rivers and streams, there is rarely uniform flow. Theoretically, a complete closed-form solution to the mathematical statement of the balance between the rate of energy loss and the rate at which it is being added by the drop in the channel bottom does not exist. Approximations, based on uniform flow analogies, provide the simplified flow relationships previously presented for steady gradually varied flow. The exactness of these approximations is a function of the accuracy of the channel geometry measurements, cross-sectional spacing, and, most importantly, an accurate estimate and use of energy losses.

(2) Other characteristics of flow in natural rivers must be considered when deciding on an approach to take for evaluating river hydraulics problems. The river engineer must also consider the effects and relative importance of the steadiness or unsteadiness of the flow and whether a one-dimensional approximation of the flow will provide sufficient accuracy and detail for the particular flow and channel configuration.

*e. Unsteady flow.* Chapter 5 presents detailed discussions regarding typical data and computer

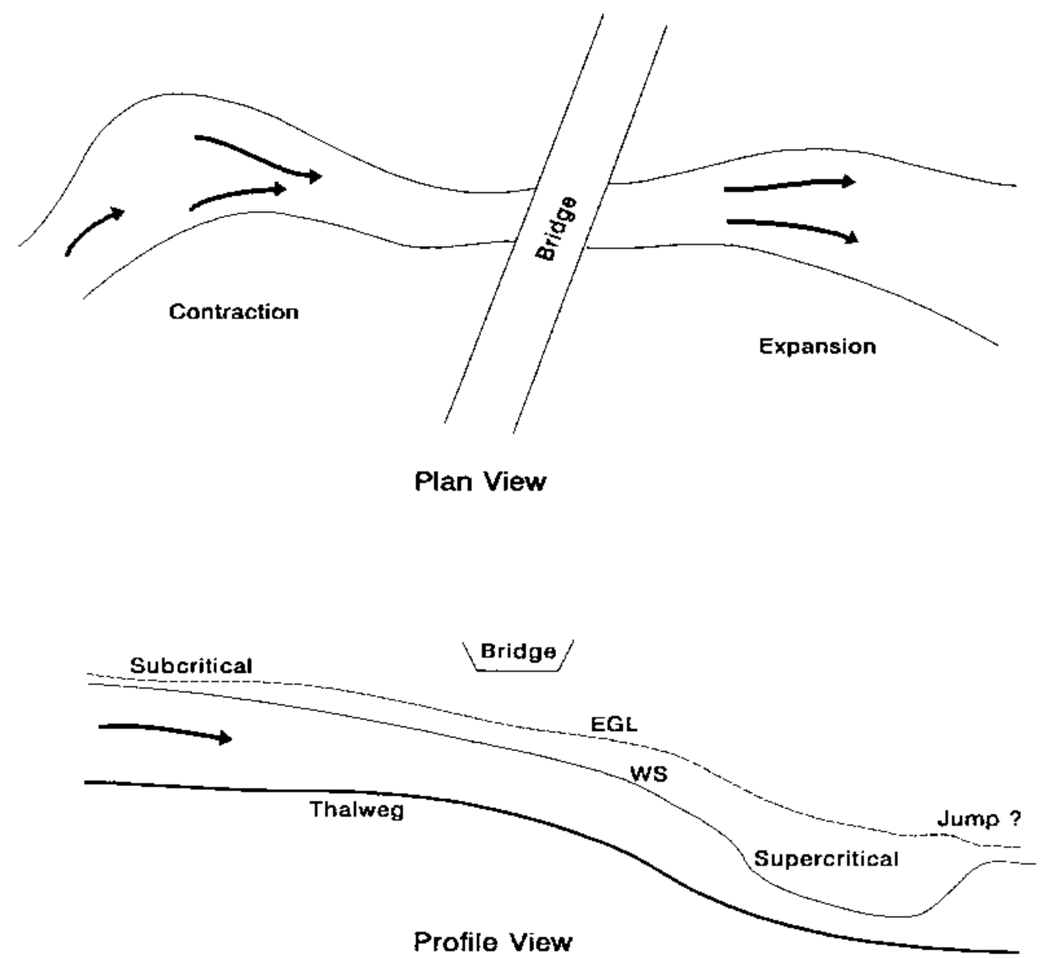


Figure 2-7. Varying flow classification along a channel

requirements as well as the various kinds and forms of hydraulic routing models that are available.

*f. Multidimensional flow.* Flow in a river channel is often considered to be one-dimensional in the direction of flow. As previously discussed, this assumption allows a simplified mathematical analysis of the flow. Multi-dimensional flows require accounting for the physics (mass and momentum conservation) of the flow in two, and sometimes three, directions. Detailed discussions of multidimensional flow analysis methods are presented in Chapter 4 and in the texts by Abbott (1979), Cunge et al. (1980), and Fischer et al. (1979).

*g. Movable boundary analysis.* Alluvial rivers often exhibit significant bed and bank mobility during and after floods. For erodible channels, use of alternative computational procedures that account for sediment transport characteristics may be necessary to accurately describe project performance with respect to channel boundary reactions and flow characteristics. Methods and procedures for evaluating alluvial channel (mobile boundary) hydraulics are presented in Chapter 7 and in EM 1110-2-4000.

*h. River channel geomorphology.* Natural streams acquired their present forms from long-term processes involving land surface erosion, stream channel incisement, streamflow variation, human activities, and land use changes. The study of these processes associated with land form development is referred to as geomorphology. In a natural river, there is a continuous exchange of sediment particles between the channel bed and the entraining fluid. If, within a given river reach, approximately the same amount of sediment is transported by the flow as is provided by the inflow, the reach is said to be in equilibrium. In natural rivers, a primary design problem is to improve, modify, or maintain the channel while also maintaining equilibrium. If a new channel is to be constructed, or an existing channel is to be altered, the primary problem is determining the stable channel dimensions.

(1) Channels may be straight, braided, or meandering depending upon the hydrology and geology of the region. The characteristics of an existing channel are a good indication of the potential success or failure of a proposed channelization project. River engineers must have some knowledge of river channel geomorphology in order to properly identify existing channel problems and to anticipate potential project-induced responses by the channel following channel modification or changing flow regulation. Texts by Leopold et al. (1964), Schumm (1977), and Petersen (1986) are excellent references. EM 1110-2-4000 also provides guidance for evaluating geomorphologic changes that can occur in rivers naturally, or as a result of human actions.

(2) The most important principle of river geomorphology that river engineers must consider is that, once disturbed, an alluvial stream or channel begins an automatic and unrelenting process that proceeds towards a new equilibrium condition. The new equilibrium characteristics (channel shape, size, depth, slope, and bed material size) may or may not be similar to the stream's original characteristics. Failure to recognize important sediment transport characteristics of an alluvial stream can lead to a situation in which a project does not perform as designed, if that design is based solely on rigid boundary hydraulics.

# Chapter 3
# Formulating Hydraulic Studies

## 3-1. Initial Considerations

When assigned a hydrologic engineering study, the tendency of many hydraulic engineers is to immediately begin the technical analysis. However, the entire study components must be planned first, recognizing the hydrologic/hydraulic information needs of other study team members. For most hydrology and hydraulics (H&H) studies, the engineer's initial effort should be spent on scoping and evaluating as many aspects of the entire study as can be identified. Besides individual experience, the hydraulic engineer should utilize the experience of others for advice and guidance in the technical aspects of the study. Frequent communications with the study manager, the economist, and other team members are necessary to ensure that their requirements are met. Other Corps personnel, the local project sponsor, and higher level reviewers will also have useful suggestions and information that will be valuable in establishing the overall scope and procedures for the hydraulic analysis. All of this information should be summarized in a written document, called a HEMP (Hydrologic Engineering Management Plan) which guides the hydraulic engineer through the course of the analysis. The HEMP is a detailed work outline covering the complete technical study. It should be the first significant item of work completed by the hydraulic engineer and should be updated during the study process as new insights are gained. The purpose of this chapter is to present the ingredients needed to develop this document. Additional information about a hydraulic work plan is given in Appendix C.

*a. Project objectives.* The objectives of a proposed project are usually broad. For the majority of Corps' work, these objectives are to provide flood control, and/or navigation to a specific reach of stream or an entire river basin. Other objectives often include hydropower, river stabilization, water supply and conservation, ground water management, permits, recreation, and environmental and water quality enhancement. For a project involving many of these objectives, the hydraulic engineer may require consultation with outside experts. Personnel from HEC, WES, the Hydrology Committee, various centers of expertise in Corps Districts, state agencies, universities, or private consultants can provide assistance in developing the hydraulic study scheme and in making decisions regarding selection of appropriate hydraulic analysis tools.

*b. Study objectives.* Once the project objectives are established, specific elements of the hydraulic analysis can be addressed. Development of the study plan requires establishment of appropriate levels of detail commensurate with the particular study phase. The appropriate level of hydraulic analysis detail is a key issue in most studies affecting, perhaps drastically, both the time and cost of the effort. This issue is often a major matter that should be resolved between the hydraulic engineer and the study or project manager early in the study.

(1) The hydraulic engineer must be knowledgeable of the planning process and design the analysis to meet the requirements of any particular reporting stage of the study (reconnaissance versus feasibility versus design). The engineer must be prepared to explain why a certain level of detail is needed, and why short-cut/less costly methods (or more expensive methods) would not (or would) be necessary and appropriate at particular stages of a study. Frequent and clear communications with the study team and development of a HEMP will facilitate specification of the appropriate levels of study detail. A justifiable H&H study cost estimate cannot be made without first developing an H&H work plan.

(2) Level of detail for the feasibility stage should be determined during the reconnaissance phase. Assuming Federal interest is found during the reconnaissance study, the most important work done in the reconnaissance report is to itemize all perceived problems and data needs and document how the study team proposes to address them in the later reporting stages. The reconnaissance report is the instrument used to define the level of detail required for the feasibility report stage. Table 3-1 overviews the objectives and level of detail typically required in the Corps' reporting process; particular circumstances may require a different blend of requirements and objectives.

## 3-2. Overview of Techniques for Conducting River Hydraulics Studies

A general overview is given below; the following chapters discuss various technical approaches in detail.

*a. Field data.* Field (prototype) data collection and analysis serves both as an important aspect of the application of other methods and as an independent method. It is an indispensable element in the operation, calibration, and verification of numerical and physical models. Also, to a limited extent, field data can be used to

**Table 3-1**
**Hydraulic Study Objectives**

| Type | Stage | Objective/Considerations |
|---|---|---|
| Pre-Authorization | Reconnaissance | Qualitative analysis: one year± time frame, primarily use existing data, with and without project analysis to determine if economic justification is likely, establish required data collection program. |
| | Feasibility | Quantitative analysis: 2-3 year time frame, with and without project H&H, economics, and plan formulation finalized, qualitative evaluation of mobile boundary problems, hydraulic design sized, continue/refine data collection program. |
| Post-Authorization | Re-Evaluation Report | Quantitative analysis: are the feasibility report findings still applicable? Update economics and hydraulics to current conditions, initiate quantitative investigation of movable boundary problems (usually). |
| | General Design | Quantitative analysis-detailed hydraulic analysis and design, detailed modeling and movable boundary analysis, finalize all hydraulics for simple projects. |
| | Feature Design | Quantitative analysis-detailed hydraulic analysis and design of one component or portion of a complex project, physical model testing, if necessary. |
| Continuing Authority | Reconnaissance Report | Qualitative analysis: usually similar to reconnaissance report portion of the feasibility report. |
| | Detailed Project Report | Quantitative analysis: a combined feasibility report and design. |

estimate the river's response to different actions and river discharges using simple computations. Obtaining detailed temporal and spatial data coverage in the field, however, can be a formidable and difficult task.

*b. Analytic solutions.* Analytic solutions are those in which answers are obtained by use of mathematical expressions. Analytical models often lump complex phenomena into coefficients that are determined empirically. The usefulness of analytic solutions declines with increasing complexity of geometry and/or increasing detail of results desired.

*c. Physical models.* Analysis of complex river hydraulic problems may require the use of physical hydraulic models. The appearance and behavior of the model will be similar to the appearance and behavior of the prototype, only much smaller in scale. Physical scale models have been used for many years to solve complex hydraulics problems. Physical models of rivers can reproduce the flows, and three-dimensional variations in currents, scour potential, and approximate sediment transport characteristics. The advantage of a physical model is the capability to accurately reproduce complex multidimensional prototype flow conditions. Some disadvantages are the relatively high costs involved and the large amount of time it takes to construct a model and to change it to simulate project alternatives. Model calibration, selection of scaling and similitude relationships, construction costs, and the need for prototype data to adjust and verify physical models are discussed by the U.S. Department of the Interior (1980), Franco (1978), Petersen (1986), and ASCE (1942). Conflicts in similitude requirements for the various phenomena usually force the modeler to violate similitude of some phenomena in order to more accurately reproduce the more dominant processes.

*d. Numerical models.* Numerical models employ special computational methods such as iteration and approximation to solve mathematical expressions using a digital computer. In hydraulics, they are of two principal types finite difference and finite element. They are capable of simulating some processes that cannot be handled any other way. Numerical models provide much more detailed results than analytical methods and may be more accurate, but they do so with increased study effort. They are also constrained by the modeler's experience and ability to formulate and accurately solve the mathematical expressions and obtain the data that represent the important physical processes.

*e. Hybrid modeling.* The preceding paragraphs described the four principal solution methods and some of their advantages and disadvantages. Common practice has been to use two or more methods jointly, with each method being applied to that portion of the study for which it is best suited. For example, field data are usually used to define the most important processes and verify a model that predicts hydrodynamic or sedimentation conditions in the river. Combining physical modeling with numerical modeling is referred to as hybrid modeling. Combining them in a closely coupled fashion that permits feedback among the models which is referred to as an integrated hybrid solution. By devising means to integrate several methods, the modeler can include effects of many phenomena that otherwise would include effects of many phenomena that otherwise would be neglected or poorly modeled, thus improving the reliability and detail of the results. A hybrid modeling method for studying sedimentation processes in rivers, estuaries and coastal waters has been developed by the Waterways Experiment Station (WES) (McAnally et al., 1984a and 1984b; Johnson et al., 1991). The method uses a physical model, a numerical hydrodynamic model, and a numerical sediment transport model as its main constituents. Other optional components include a wind-wave model, a longshore current calculation, and a ship handling simulator.

*f. Selection of procedure.* Tables 3-2 and 3-3 give suggestions, based on experience, regarding usage of the various procedures in different phases of flood control and navigation studies. This information should be viewed as a starting point; it will change as computer resources and the Corps' planning process and missions evolve.

**Table 3-2**
**Model Usage During Hydraulic Studies For Flood Control Projects**

| Stage | Existing Data & Criteria | GVSF | MB | GVUSF | Multi-D | Phys.* |
|---|---|---|---|---|---|---|
| Reconnaissance | X | X | ?(1) | | | |
| Feasibility | | X | X(1) | X(2) | ? | ? |
| Re-evaluation | | X | X | X | ? | ? |
| General Design Memo. | | X | X | X | X(3) | X(3) |
| Feature Design Memo. | | | | | X(3) | X(3) |
| Continuing Authority | X | X | X(1) | ? | ? | ? |

* Existing Data and Criteria = available reports, Corps criteria, regional relationships for depth-frequency, normal depth rating relationships, etc.; GVSF = gradually varied, steady flow [i.e. HEC-2, HEC (1990b)]; MB = mobile boundary analysis [i.e. HEC-6, HEC (1991a)]; GVUSF = gradually varied unsteady flow [i.e. UNET, HEC (1991b); not including hydrologic models like HEC-1, HEC (1990a)]; Multi-D = multidimensional analysis [i.e. TABS-2, Thomas and McAnally (1985)]; Phys. = physical models (by WES or similar agency).

? Possible, but very unusual - highly dependent on problem being analyzed.

(1) Sediment problems must be addressed, but the procedure at this stage may be qualitative or quantitative, depending on the type and magnitude of the project.

(2) Use is possible, but unlikely, on most flood control studies.

(3) Typically employed to evaluate design performance for a short reach of river or in the immediate vicinity of a specific project component, or to refine the hydraulic design of a project component.

**Table 3-3**
**Model Usage During Hydraulic Studies For Navigation Projects**

| Stage | Existing Data & Criteria. | GVSF | MB | GVUSF | Multi-D | Phys.* |
|---|---|---|---|---|---|---|
| Reconnaissance | X | X | | | | |
| Feasibility | | X | X(1) | ? | ? | ? |
| Re-evaluation | | X | X | ? | ? | ? |
| General Design Memo. | | | X | X | X | X |
| Feature Design Memo. | | | | | X | X |
| Continuing Authority | X | X | X(1) | (2) | (2) | ? |

* As defined in Table 3-2.

? As defined in Table 3-2.

(1) Sediment problems must be addressed at this stage, either quantitatively or qualitatively. Detailed movable boundary analysis with computer modeling is more likely at this stage for a navigation project than for a flood control project.

(2) Navigation projects for this stage are typically small boat harbor or off-channel mooring facilities of rather uncomplicated design. GVUSF or multidimensional modeling techniques are normally not utilized. A field survey during the reconnaissance and data gathering stages of a study by the responsible hydraulic engineer is essential.

## 3-3. Analysis of Hydraulic Components

Most problems that are studied have solutions that include hydraulic structures that are identified early in the reconnaissance phase. Different types of structures require different methods for proper evaluation. General guidance for method selection is given in Table 3-4 for flood control, navigation, and hydropower projects. The study objectives, along with the type of hydraulic component to be evaluated, should indicate the type of analysis required.

## 3-4. Data Requirements

There are three main categories of data needed for hydraulic studies: discharge, geometry, and sediment. Not all of these categories, or all of the data within each of these categories, will be needed for every study.

*a. Discharge.*

(1) A project is usually designed to perform a function at a specific discharge. It must also function safely for a wide range of possible flows. Flood control projects are usually designed for the discharge corresponding to a specific flood frequency, or design event, while navigation studies use a discharge for a specific low flow duration or frequency. The single discharge value for the hydraulic design should not be over-emphasized; rather, project performance must be evaluated for a range of flows, both greater than and less than the "design discharge." A levee may be designed to provide protection from the one-percent chance flood, but the levee design must also consider what happens when the 0.5- or 0.2-percent chance or larger flood occurs. A channel may be designed to contain the 10-percent chance flood, but the annual event may be the most dominant in terms of forming the channel geometry to carry the stream's water/sediment mixture. In some cases, the absence of a low flow channel to carry the everyday water and sediment flows has caused the 10-percent chance channel to be quickly silted up. Similarly, steady flow evaluations may be insufficient to adequately evaluate project performance. Full hydrographs or sequential routings for a period of record may be required to address the project's response to sediment changes or the occurrence of consecutive high or low flow periods. Velocities are important for water quality, riprap design, and other engineering studies. Velocity for the peak design flow

**Table 3-4**
**General Guidelines for Typical Methods of Analysis for Various Hydraulic Components**

| Flood Control Component | Typical Analysis Procedures |
|---|---|
| Levees | GVSF normally; sediment analysis: often qualitative, but detailed movable boundary analysis may be necessary on flank levees. |
| Dams (height) | Normally hydrologic reservoir routing, or GVUSF. |
| Spillways | As above to establish crest elevation and width, general design criteria from existing sources to develop profile, specific physical model tests to refine profile. |
| Stilling Basins | General design criteria from existing sources to establish floor elevations, length and appurtenances, specific model tests to refine the design, movable boundary analysis to establish downstream degradation and tailwater design elevation. |
| Channel Modifications | GVSF normally, qualitative movable boundary analysis to establish magnitude of effects, quantitative analysis for long reaches of channel modifications and/or high sediment concentration streams, physical model tests for problem designs (typically supercritical flow channels). |
| Interior Flood | Integral part of a levee analysis - hydrologic routings normally for pump and gravity drain sizing, GVSF for ditching and channel design, physical model testing for approach channel and pump sump analysis. |
| Bypass/ Diversions | GVSF or GVUSF analysis, physical model testing, movable boundary analysis on sediment-laden streams. |
| Drop Structures | Similar to stilling basin design, although model tests often not required. |
| Confluences | GVSF usually, GVUSF for major confluences or tidal effects. |
| Overbank Flow | GVSF normally, GVUSF/Multi-D for very wide floodplains or alluvial fans. |
| FPMS Studies | GVSF normally. |
| **Navigation** | |
| Channel Modifications | Dikes - Movable boundary analysis (quantitative), multidimensional modeling, physical model tests. |
| | Cutoffs - GVSF or GVUSF, movable boundary analysis to establish the rate of erosion and channel shifting, physical modeling. |
| | Revetment - general design criteria from existing sources, GVSF, physical model tests. |
| Navigation Dams | Normally, GVSF to establish pool elevations, profiles and depths, multidimensional modeling to estimate current patterns, physical model testing, movable boundary analysis to establish downstream scour for stilling basin design. |
| Locks | General design criteria from existing sources, possible multidimensional modeling/physical modeling for approach and exit velocities and refinements of lock design and filling/emptying systems. |
| **Other** | |
| Hydropower | System simulation for optimal operation. Multidimensional analysis for flow patterns, physical model tests. |

or velocities for specific time periods may be needed, depending on the study requirements.

(2) Discharge data include measured and/or synthesized flows along with frequency, velocity, duration, and depth information. Measured data at gages are the preferred source for this category; seldom, however, does sufficient measured data exist. A typical hydraulic analysis requires simulated data from hydrologic models as well as information on historical events, usually floods. This latter data is often obtained from extensive discussions with local residents living along the study stream and the review of newspaper accounts and/or Corps or other agency reports. A field survey during the reconnaissance and data gathering stages of a study by the responsible hydraulic engineer is essential.

*b. Channel geometry.*

(1) Channel geometry is required for any hydraulic study. Geometric data include channel and overbank topography, stream alignment, bridge and culvert data, roughness information, changes in stream cross section shape, and alignment over time. Extensive field and/or aerial surveys supply the bulk of these data; however, cost reductions can be achieved by locating and using available data. Most rivers and streams have been studied in the past. Floodplain or flood insurance reports are often available and can be valuable sources of geometric and other data. Bridge plans are usually available from state, county, or municipal highway departments. Navigable rivers have hydrographic surveys of the channel taken periodically. Aerial photos have been taken at regular intervals by the Soil Conservation Service since the mid-1950's providing data on stream channel changes. Even if it is decided that new surveys need to be obtained, the above sources provide valuable information on changes in channel alignment and geometry over time, indicating potential problems related to the stream's sediment regime. The keys to the usefulness of the data are the accuracy of the survey data and the locations of cross sections along the stream. Accuracy is discussed in section 3-4*e* and Appendix D. Additional information on the effects of survey data accuracy on computed water surface profiles can be found in "Accuracy of Computed Water Surface Profiles" (USACE 1986).

(2) The amount of survey data required depends on the study objective and type. For instance, more frequent surveys are needed for navigation projects than for flood control projects. Detailed contour mapping for urban studies should be obtained in the feasibility phase rather than in the design phase, whereas detailed mapping for agricultural damage reduction studies may often be postponed to the post-authorization stage. For movable bed studies repeat channel surveys are needed at the same locations, separated by significant time periods, to evaluate a model's performance in reproducing geometric changes. Thalweg profiles and/or repetitive hydrographic surveys are needed for analysis of bed forms and the movement of sand waves through rivers.

*c. Sediment.*

(1) The amount of sediment data needed is not always apparent at the beginning of a hydraulic study. The sediment impact assessment, as outlined in EM 1110-2-4000, is performed during the initial planning process. Sediment assessment studies are typically performed to determine if the project proposal is likely to create a sediment problem or aggravate an existing one. The results of this evaluation will dictate the need for additional data and quantitative studies during the feasibility and design phases. If a sediment problem presently exists, or is expected with a project in place, a sediment data collection program must be initiated so that the problem can be properly addressed in later stages of the analysis.

(2) Sediment data include channel bed and bank material samples, sediment gradation, total sediment load (water discharge versus sediment discharge), sediment yield, channel bed forms, and erosion-deposition tendencies. Long-term sediment measuring stations are few in number, and modern methods of sediment measurement can make older records questionable. Sediment data collected at a gaging site are usually short-term. Flood control or navigation studies must address sediment to determine if there is, or will be, a sediment problem if the study proposal is implemented. Often, the initial sediment analysis is performed in a rather qualitative fashion with a minimum amount of data. If there appears to be a sediment problem, a data collection program should be established, at least for a short period, to obtain calibration data. Chapter 7 and EM 1110-2-4000 should be reviewed for further guidance on sediment data.

(3) The type of project often dictates the amount and type of sediment data needed. For instance, reservoir and channelization proposals require that the entire suspended sediment load (clays, silts, sands, and gravels) be analyzed, whereas flood control channels or river stabilization projects primarily require analysis of the bed material load (mainly sands and gravels) because the finer materials (clays and silts) usually pass through the

reach. The latter type of projects may require less data than the former. For example, an evaluation of the bed material at and near the surface, through "grab samples" or collection with hand augers, may be adequate. If the material consists of fine sands, a detailed sediment study may be required, possibly in the feasibility phase.

*d. Data availability.* Data are usually available from the U.S. Geological Survey's (USGS) nationwide data collection system. Corps' water data measurements provide another source; in many parts of the United States state agencies and water conservancy districts also collect water data. If measured data are not available but are required for the study, a data collection system is necessary. Guidance on specifying and developing a gaging system is available from the USGS (1977) with additional information in ER 1110-2-1455. Definition of the need for certain data and budgeting for its collection should be included in the feasibility or reconnaissance report cost estimates.

*e. Accuracy of data.* Results from numerical models are routinely available to a precision of 0.01 foot, implying far more solution accuracy than that of the basic data. The hydraulic engineer should be aware of the impact of input data uncertainty relative to reliability of the computations. There are relatively few USGS discharge gages having records rated as "excellent." This rating carries an explanation that 95 percent of the daily discharge values are within 5 percent of the "true" discharge (thus 5 percent are outside of that limit). "Good" records have 90 percent of the daily discharges within 10 percent. If any specific discharge varies by 5 percent, the corresponding stage could vary significantly depending on the stream slope and geometry. Instantaneous peak discharges presumably would be less accurate. Thus, a potentially significant accuracy problem exists with the basic data.

(1) Geometric data are more accurate than flow data; however, some variation is still present, see U.S. Army Corps of Engineers (1989). If not located properly, cross sections obtained by any technique may not be "representative" of the channel and floodplain reach for which each section is used (see Appendix D). Significant errors in water surface profile computations have occurred when distances between cross sections were large. Closer cross section spacings will improve the accuracy of the profile computations (i.e. the solution of the equations), but will not necessarily result in a better simulation unless the sections are properly located to capture the conveyance and storage in the reach. A more detailed discussion of river geometry requirements is provided in Appendix D. The computer program "Preliminary Analysis System for Water Surface Profile Computations (PAS)" is designed to assist with data development for profile computations (U.S. Army Corps of Engineers 1988b).

(2) Sediment data have the most uncertainty, due both to the difficulties in obtaining the measurements and the incorporation of discharge and geometry measurements in the calculation of sediment load. Sediment load curves typically are the most important relationships in sediment studies. This water discharge/sediment discharge relationship should be sensitivity tested to evaluate the consequences of an over- or under-estimate.

(3) Absolute statements as to the accuracy of final hydraulic results should be tempered by an understanding of the field data accuracy. The more accurate the final hydraulics are required to be, the more accurate the data collection must be. Sensitivity tests to evaluate possible over- or under-estimates should be routinely made.

*f. Hydraulic loss coefficients.* Various energy loss coefficients are required for hydraulic studies. These energy loss coefficients include channel and overbank friction, expansion-contraction losses, bridge losses, and miscellaneous losses.

(1) Manning's *n*. For the majority of hydraulic studies, Manning's *n* is the most important of the hydraulic loss coefficients (U.S. Army Corps of Engineers 1986). The variation of water surface elevation along a stream is largely a function of the boundary roughness and the stream energy required to overcome friction losses. Unfortunately, Manning's *n* can seldom be calculated directly with a great deal of accuracy. Gage records offer the best source of information from which to calculate *n* for a reach of channel near a gage. These calculations may identify an appropriate value of *n* for the channel portion of the reach. Whether or not this value is appropriate for other reaches of the study stream is a decision for the hydraulic engineer. Determination of overbank *n* values requires a detailed field inspection, reference to observed flood profiles, use of appropriate technical references, consultation with other hydraulic engineers, and engineering judgment. For some streams, *n* varies with the time of year. Studies on the Missouri (U.S. Army Corps of Engineers 1969) and Mississippi Rivers have found that Manning's *n* is significantly less in the winter than in warm weather for the same discharge. If stages are to be predicted in the winter as well as the summer, temperature effects must be addressed. Similarly, many sand bed streams demonstrate a great

change in bed forms as discharge increases. A threshold level exists such that when discharge and velocity reach a certain range, the bed changes from dunes to a flat bed, thus dramatically decreasing *n*. A higher discharge can pass at a lower elevation than an earlier, lower, discharge due to this phenomena. This "discontinuous" rating curve is a characteristic of many streams. An example is shown in Figure 3-1. References by Chow (1959), French (1985), and Barnes (1967) may be used to assist in the estimation of *n* for a reach of stream. A more complete discussion of loss coefficients is provided in Appendix D.

(2) Equivalent roughness, *k*. An alternate method of defining Manning's *n* is by estimating an equivalent roughness coefficient *k*. This technique is described by

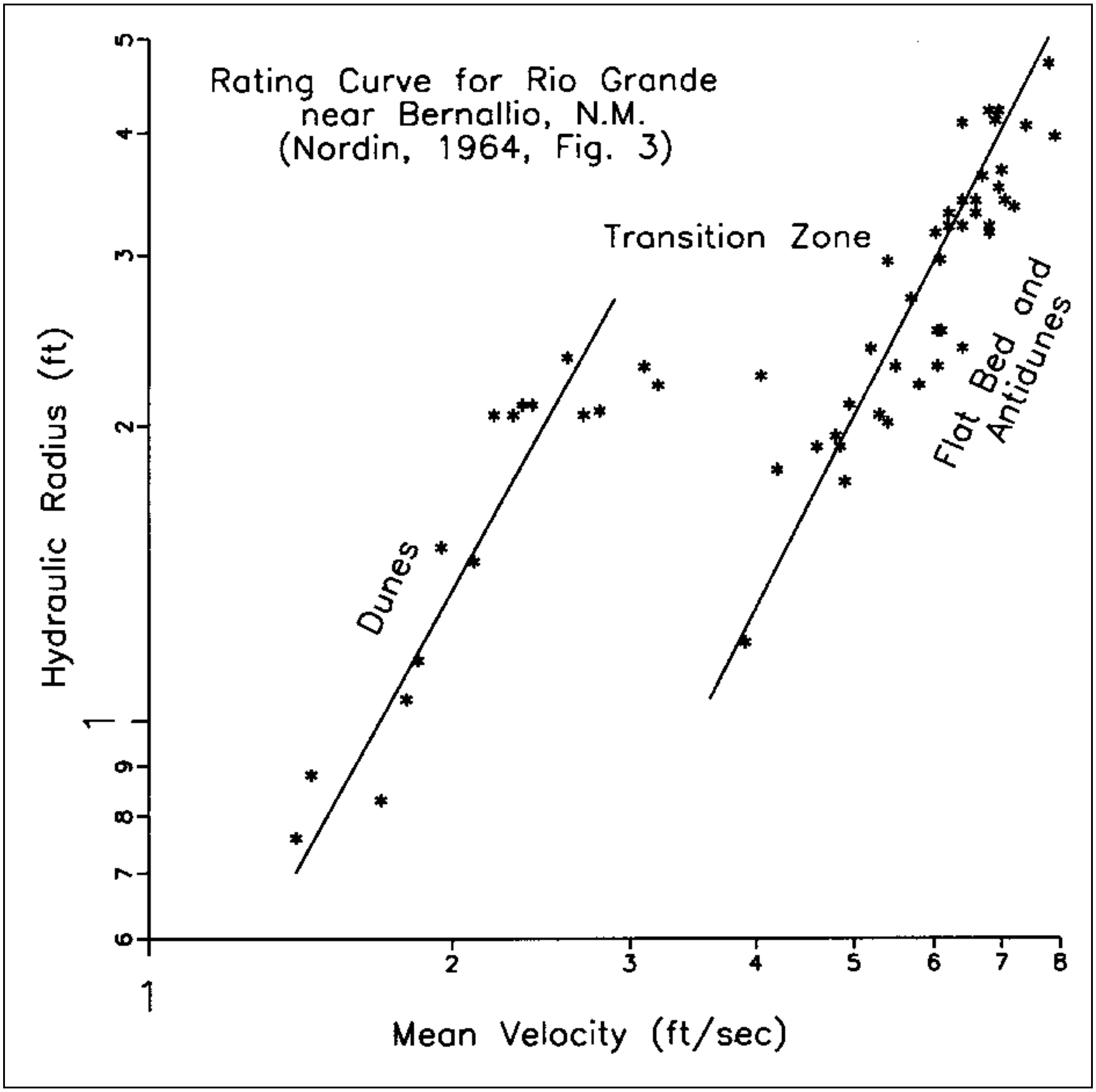


**Figure 3-1. Discontinuous rating curve**

Chow (1959) and in EM 1110-2-1601. It relates *n* to a function of *k* and the hydraulic radius (R). A *k* value is the equivalent diameter, in feet, of the predominant grain size in the channel or the average size of an overbank obstruction. Advantages to using *k* to calculate *n* include adjustments to *k* as depth changes are not required; *n* can be found directly from *k* and the R for the stage being evaluated, and errors in estimating *k* result in only small differences in the calculated value of *n*. The engineer must evaluate the significance of other factors influencing *n*, including bed form changes, channel alignment, cross-sectional area changes, and bank vegetation. Field inspection of the study stream at varying states of flow is imperative for attaining appropriate estimates of *n* for ranges of discharge. It is not beyond reason to expect the hydraulic engineer to walk or float the entire reach of stream to determine friction values.

(3) Expansion-contraction coefficients. Although water surface profiles are mostly influenced by friction forces, changes in the energy grade line, and the corresponding water surface elevations can result from significant changes in stream velocity between cross sections. This is most apparent in the vicinity of bridges which tend to force the discharge through an opening smaller than the upstream and downstream channels. Therefore, a contraction into and an expansion out of a bridge results in eddy energy losses. These losses are usually quantified with coefficients of expansion or contraction (when using a one-dimensional approach), based on the abruptness of the change. For most situations, the expansion/contraction energy losses are not great except in the vicinity of bridges and culverts. Using the appropriate coefficient at each streamflow obstruction is important, as well as adjusting the coefficient back to an appropriate value upstream of the obstruction. The references by Chow (1959) or U.S. Army Corps of Engineers (1988a, 1990b) provide typical values of expansion and contraction coefficients.

(4) Bridge losses. Bridges that cause relatively small changes in the energy grade and water surface profiles can be adequately modeled using appropriate values of Manning's *n* and expansion-contraction coefficients. Bridges that cause the profile to become rapidly varied near and within the bridge require other methods of analysis. Weir flow over the roadway, pressure flow through the opening, and open channel flow where critical depth in the bridge occurs are examples where detailed bridge analysis is required. To correctly model losses for these situations, bridge geometry becomes more important. The number, location, and shape of bridge piers must be obtained; a roadway profile and weir coefficient are needed for weir flow calculations; guardrails and/or bridge abutments which serve to partially or fully obstruct weir flow must be defined; the precise upstream and downstream road overtopping elevations must be identified (often through trial and error computations) and debris blockage estimated. Photographs and verbal descriptions of each bridge and field dictated to a hand-held tape recorder are most useful when modeling each bridge. References by U.S. Army Corps of Engineers (1975, 1988a, 1990b) should be consulted for additional information.

*g. Study limits.* The appropriate spatial scope for a hydraulic study is often incorrectly identified, particularly if all possible project effects are not envisioned. The study, or model, should not start and stop at the physical limits of the proposed project. Rather, the boundaries should extend far enough upstream and downstream from the project limits to completely encompass the full effects of the project on the basin. Reservoir, channelization, levee, and navigation projects may produce changes in stage, discharge, and sediment conditions that can affect reaches well removed from the physical location of the project. For example, major channelization, resulting in shortening of the stream, may generate upstream headcutting and downstream deposition that can continue for decades. Reservoirs can cause upstream deposition, thereby increasing water surface elevations over time, and may cause downstream degradation because of the relatively sediment-free waters that are released. The deposition and degradation can extend up tributaries also. Study limits must be established so that all effects of the project, both positive and negative, can be identified and evaluated. Figure 3-2 illustrates some considerations for establishment of study limits for a reservoir project and the type of data required at various locations within the study area.

*h. Possible needs for additional data.* Not all data needs can be foreseen at the start of a study. Consultations with experienced personnel early in the study are often useful in identifying data needs. Some common needs that often surface well into a study include stage and/or discharge duration data (especially where stage-frequency near a stream junction becomes important), surficial soils analysis to estimate sediment yield for ungaged areas (particularly where the amount of sand compared to the amount of fines is important), type and gradation of bed material present at different times for movable bed model calibration, measurement of velocity directions and magnitudes at various stages, times, and locations for use in multidimensional model calibration.

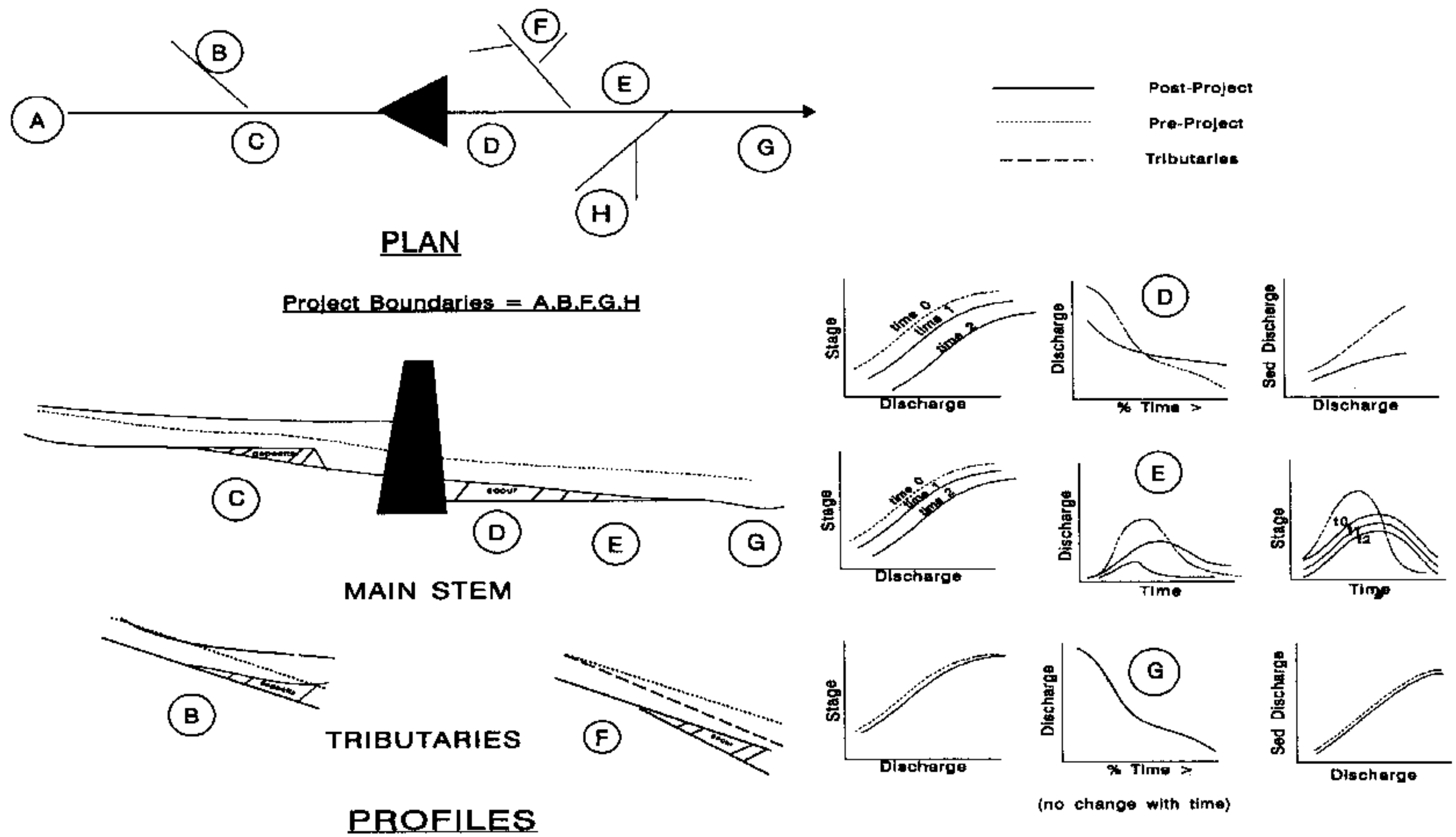


Figure 3-2. Example study limits and possible project impacts

*i. Other factors.* Ongoing or near-future, changes in the watershed should be considered in developing water surface elevations. Consideration of urbanization effects on future discharges has long been a requirement of Corps analysis. Other localized effects should also be considered. Local channel modifications and bridge replacements that are ongoing or scheduled to be completed prior to implementation of a Corps project should be incorporated into the hydraulic study. Bridge obstructions, particularly culverts under a high fill, can cause significant upstream ponding and induce damage to nearby structures. If the local community has no plans (or funds) to rectify a severe local flooding problem such as this, the Corps study team should include this obstruction in the future condition, without project, analysis. On several occasions, however, in the time between the Corps' feasibility report and the final design document, such obstructions have been replaced, greatly decreasing project benefits and affecting the authorized plan. Sensitivity tests on economic effects to the Corps' recommended plan of potential modifications to culverts or bridges are encouraged. The project manager should maintain continuous contact with the local community and highway department to obtain information on potential bridge replacements that may affect the project.

## 3-5. Calibration of Hydraulic Analysis Models

The reliability of the results of a hydraulic model study depends on the skills and experience of the hydraulic engineer performing the study, applicability of the model to the physical situation, and the quality of the data used to both model the study reach and calibrate the model. The overall calibration process incorporates three distinct steps: obtaining the necessary data and translating it into input for a numerical model, calibrating the model, and verifying the model. Additional guidance on calibration is given in Chapters 4 through 7 and Appendix D.

*a. Purpose of calibration.* The objective of the calibration process is to match the output of the model with observed data (usually water surface elevations). This process is performed by adjusting one or more parameters, such as Manning's *n*, until a satisfactory match of model results with known data is achieved. When a set of known conditions has been approximately matched by the model, one can apply the model to unknown conditions (the 1-percent chance flood, the Standard Project Flood, etc.) with more confidence that the model output is reasonably representative of the physical processes associated with that event. However, to be confident, the observed data for calibration should be obtained from an event that is near the scale of the events to be modeled.

*b. Observed data.* This includes data recorded at gages along with that obtained from field observations by Corps personnel, and from interviews with local residents. Recorded discharges, stages, and velocities are valuable for calibration purposes; however, it is rare that sufficient gage data are available for comprehensive calibration. The preponderance of calibration data usually comes from local observations during and after an event. The hydraulic engineer should plan for several days of field work to obtain highwater marks from local residents' observations or following an event that occurs during the study. The best data often come from people who have lived near the stream for many years. They can supply information concerning flood elevations, erosion or deposition tendencies, local channel modifications (when and where), tendencies for debris to obstruct bridge openings, how often the stream gets out of banks, and possible flow transfers between watersheds during floods. As much information as possible should be obtained from local residents for use in the calibration process. While all information is useful, the hydraulic engineer should recall that the further back in time, often the hazier the memory of the individual is for exact flood heights. The exact water level of the flood may not be accurately recalled. The engineer should not expect that model results will match every highwater mark exactly.

*c. Calibration process.* The calibration process normally focuses on matching stage and discharge data at gaging sites with highwater marks used to calibrate the model at ungaged sites. This section addresses only the stage or highwater mark calibration.

(1) The first step in the process does not begin until the study reach data have been assembled and entered into an input file, several discharges have been simulated, and the data file corrected as necessary. Effective flow area transitions between adjacent cross sections should be reasonable; profiles through bridges should be closely inspected to ensure that faulty modeling procedures are not leading to incorrect head losses and computed water surface profiles; and all warnings or messages from a numerical model should be reviewed and corrected if necessary. The hydraulic engineer should ensure that the model is performing reasonably well before "fine tuning" is initiated to match model results to field data.

(2) For subcritical flow, one-dimensional steady flow water surface profile computations begin

downstream from the study reach, preferably at a reliable boundary condition. If starting conditions are not known, the engineer must ensure that profile computations begin sufficiently far downstream that any errors in estimating starting water surface elevation will be eliminated by profile convergence to the correct elevation downstream of the study reach. This distance is mainly a function of the stream slope. Additional guidance on selecting the correct distance downstream of the study reach is given in "Accuracy of Computed Water Surface Profiles" (U.S. Army Corps of Engineers 1986).

(3) The channel *n* value can be calibrated for various flows if stage-discharge data are available (e.g. at a gage). Once a match of computed and actual stages at a gage site for in-bank flows is obtained, the channel *n* may be held constant and the overbank *n* calibrated for different historic floods. For one or more known discharges, the computed profile should be plotted and compared with measured stages and highwater marks. It should not be expected that the two will exactly coincide. A successful calibration occurs when the computed profile is close to the majority of highwater marks, with some scatter allowed. Means to achieve a calibration include changes to Manning's *n*, adjustments to expansion/contraction coefficients where warranted, modifications to effective flow boundaries, or to bridge geometry descriptions. Typically, most of the adjustments are to Manning's *n*.

(4) Considerable uncertainty exists in the estimation of *n*, with estimates by experienced hydraulic engineers commonly differing by ±20 percent at the same stream section (U.S. Army Corps of Engineers 1986). Thus, one can reasonably justify an increase or decrease of this magnitude to calibrate a model. The hydraulic engineer should be cautious if an "unreasonable" adjustment to *n* is required for calibration. Rigorous guidance on acceptable calibration errors cannot be given. The judgment and experience of the responsible hydraulic engineer and reviewers is foremost. Rules of thumb of ± 1 foot are often used, but this criterion may not be acceptable for all situations, particularly for steep streams. Some general considerations for the calibration process are given in Table 3-5. Figure 3-3 shows an example of satisfactory water surface elevation calibration for a stream reach. The process and rationale for calibration should be documented in the study reports.

(5) Additional calibration data are necessary for the application of two-dimensional, unsteady flow, and sediment transport models. Each chapter on the application of the various methods provides information on model calibration and verification.

*d. Verification.* The last step in the calibration process is verification of the model. This operation is most desirable, but is not always possible, often requiring more data than is available. The verification process is

**Table 3-5**
**Data Gathering/Calibration Considerations**

- Obtain as many highwater marks (HWM) as possible after any significant flooding, no matter how close together and how inconsistent with nearby HWM's. Physically describe each HWM location so that surveys may be obtained at a later date.
- Obtain highwater marks upstream and downstream of bridges if possible, so that the effects caused by these obstructions can be estimated and so that bridge modeling procedures may be confirmed.
- Check on bridge/culvert debris blockages with local residents. For urban streams, check with residents and newspaper files on occurrences of bridge opening blockages by automobiles or debris.
- For historical flooding, check on land use changes, both basin wide and local, since the flood(s) occurred.
- What has been happening to the stream since the last flood? Erosion or deposition that may have occurred since historic floods, if significant, will render calibration with today's channel configuration invalid.
- If HWM's are taken from debris lines, remember that wave wash can result in the debris line being higher than the HWM, particularly for pools.
- Is the observer giving the HWM biased? A homeowner may give an exaggerated HWM if the owner thinks it might benefit a project; the owner with a house for sale may give a low estimate or indicate no flooding occurs if he/she thinks it will affect the sale.

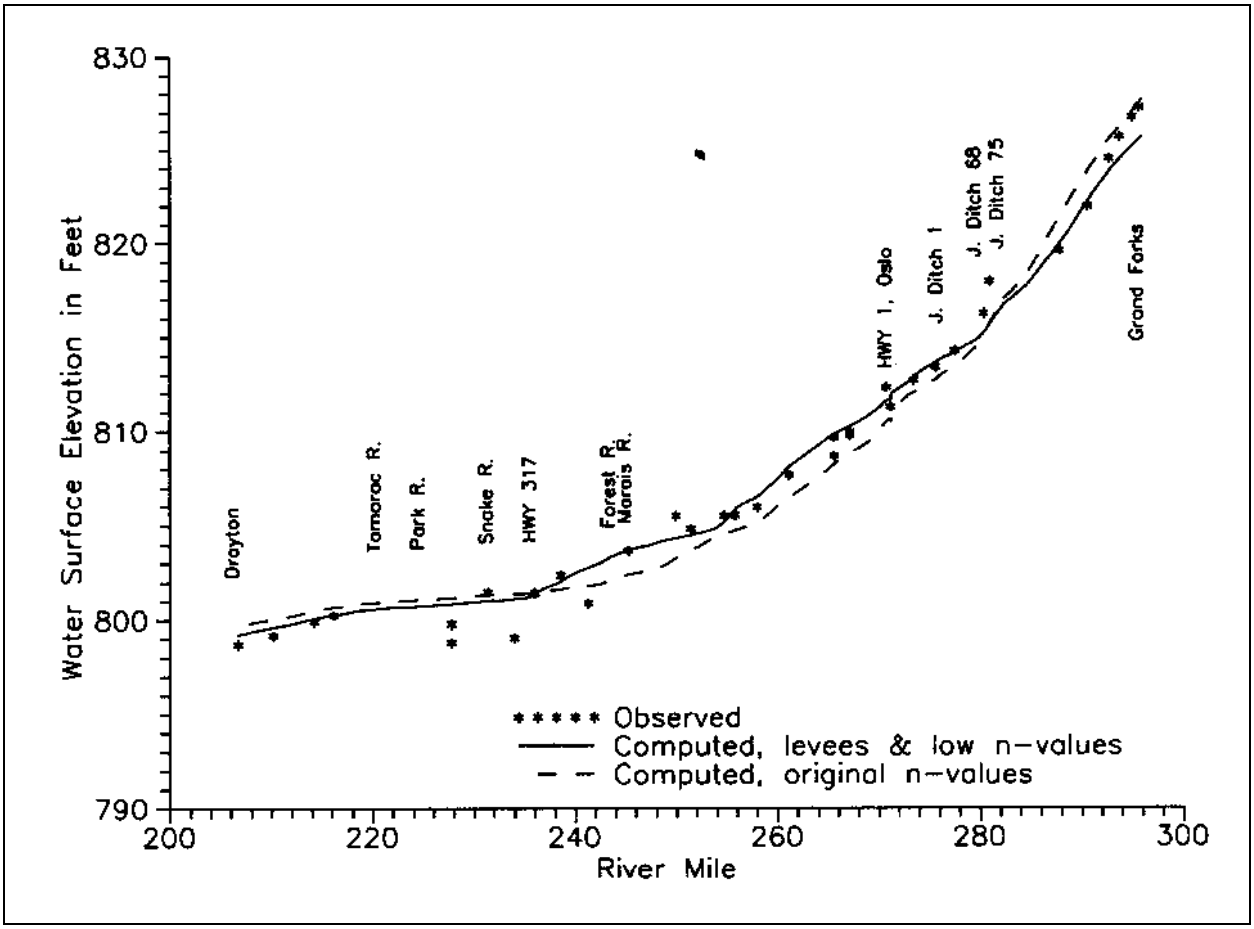


**Figure 3-3. Profile calibration to high water marks**

similar to the "split sample" testing procedure of frequency analysis. The calibrated model is used to compute elevations from additional flood events that were not used during the calibration process. The objective of this test is to confirm that the calibrated model can be used with confidence for other events. If only one or two floods have data, insufficient information may exist for the verification process; however, the verification step should be part of the overall calibration process. In the absence of data for verification, additional sensitivity analyses should be performed to evaluate the potential range of results due to uncertainty in input data.

## 3-6. Guidelines for Analytical Model Selection

The choice of appropriate analytical methods to use during a river hydraulics study is predicated on many factors including (1) the overall project objective, (2) the particular study objective for the project (level of detail being called for), (3) the class, type, and regime of flows expected, (4) the availability of necessary data, and (5) the availability of time and resources to properly address all essential issues. The following sections discuss the importance of these factors.

*a. Study objectives.* The type of analytical model selected by the hydraulic engineer should reflect the demands and objectives of the study. The type of model required may not be apparent until the hydraulic engineer becomes well-versed in the problems to be evaluated and spends considerable time with the study manager, economist, and local sponsor, discussing problems and potential solutions. Much of the initial reconnaissance work focuses on this problem. The level of detail relates directly to the model selected, as was described in section 3-1*b*. The study manager or local sponsor may specify or request a certain level of detail that may or may not be appropriate for the stage of the study. The hydraulic engineer must be able to designate the level of detail required for the problems to be studied, stage of the study, and intelligently discuss these requirements with the study manager, and local sponsor. It is the responsibility of the hydraulic engineer to ensure that the level of detail is not too little nor too much for the stage of the study.

(1) Although absolutes cannot be given regarding the level of detail for specific studies, Table 3-2 gives some representative guidance. In general, gradually varied steady flow is appropriate for most feasibility report analyses. Exceptions include those projects that obviously have an extensive effect on sediment regime (major channelization or reservoirs) that require movable boundary analysis in the feasibility phase, or those projects that may significantly change velocity patterns or cause rapid changes in stage (locks and dams, power plant operations, etc.). Movable bed models and unsteady or multidimensional models are often utilized in the design stage, often after a data collection program has been in place to obtain the necessary data with which to calibrate and verify these more complex models.

*b. Data availability.* While the first consideration should be study stage and level of detail required, the amount of available data also plays a part in the model selection. Gradually varied steady flow models can be calibrated with only highwater marks whereas movable boundary and unsteady or multidimensional models may require data from the entire hydrograph to calibrate. These models also require more hydraulic engineer skill and computer resources than gradually varied steady flow models. The necessity of using more sophisticated models will usually become apparent in the planning process. Occasionally, higher level models must be used in the survey report stage, even without adequate calibration data. While the level of reliability may suffer due to limited or no calibration data, a skilled and experienced hydraulic engineer should be able to utilize such models to evaluate changes or differences due to a project, even though absolute with or without project values are questionable. If accuracy is critical to the results of the feasibility report, a data collection program must be budgeted and planned for during the reporting process.

*c. Accuracy considerations.* The term "accuracy" is rather nebulous when applied to hydrologic engineering. Physical and numerical models can yield information with a high level of precision, but with accuracy limited by the input data. The field data used to develop, calibrate, verify, and operate models often vary ±10 percent, or more, from the actual values.

(1) The best evidence of the accuracy of the results is the skill and experience of the hydraulic engineer performing the analysis. Rather than specifying a numerical range, an appropriate reply to an accuracy question might be: "Because the model has adequately reproduced known events, the results for other, hypothetical, events are deemed to be representative of what would occur and results can be used with a reasonable level of confidence, provided that the same physical processes dominate in both known and hypothetical events." Implied in the foregoing is the use of sensitivity tests to evaluate the influence of key variables (like *n* values) on design profiles to judge the sensitivity of project economics to those profiles.

(2) Determination of existing condition profiles requires the most care in the feasibility stage, as these profiles are key in the evaluation of existing potential damages, and flood hazard. Design studies require more accuracy for designing hydraulic components than necessary in the feasibility stage.

*d. Modeling requirements (time, experience, and computer resources).* Modeling requirements vary with the reporting stage. In general, the more sophisticated the model required, the more time and cost is involved and the more limited is the pool of experienced engineers from which to draw. Only one or two experienced hydraulic engineers (at most) are usually available in any office to perform a hydraulic study requiring a multidimensional or movable boundary model. Other hydraulic engineers can encounter considerable start-up time and cost due to their inexperience with these techniques.

*e. Hydraulic considerations.* Computation of flow characteristics in natural channels can be a complicated and difficult task. Many design failures and maintenance problems have resulted from the application of inadequate or inappropriate analytical methods for the problem being considered. It is essential, therefore, to choose, develop, and calibrate the proper analytical method or modeling approach from the very beginning of a river hydraulics study. Much of the success of a project evaluation lies in the ability to properly formulate the hydraulic studies as one of the first tasks performed by the study team. The type of analysis needs to be accurately defined prior to selecting the model so that the study objectives dictate the model usage and not the other way around.

(1) As overviewed in Chapter 2, the classification and state of flow should be estimated as best as possible as an aid in selection of an analytical tool. Considerations are:

- Flow Classification: Open channel, Pressure, or Both

- Flow Type: Steady - gradually or rapidly varied Unsteady - gradually or rapidly varied

- Locations of Controls: Subcritical reaches, supercritical reaches, transitions, structures, rating curves

- Boundary type: Fixed or mobile

*f. Other considerations.* Once the study objectives, funds, study time frame, data and personnel availability are determined, several other important questions and considerations should be made prior to selecting a particular numerical or physical model. These may include:

- Are the data requirements of the model consistent with the study objectives? Personnel costs are usually more significant than computer costs.

- Capacity of the model and available computer hardware and software to provide information required for the study.

- Adequacy of the theoretical basis of the numerical model.

- Degree to which the model has been tested and verified.

- Data requirements in relation to data availability and amount of pre-processing required. Also, are the available data proprietary or public?

- Ease of application of the program. Factors include model documentation, input structure, diagnostic capabilities, output structure, flexibility to display output, and support.

- Data management capabilities (e.g., ability to pass information from one module to another).

- Ease of making program modifications, either in-house or by contract.

- Program efficiency in terms of typical run times and costs.

- Program accessibility. Can the program be run on a computer that is convenient to access? Does it require a mainframe computer or special hardware?

- Accessibility of user-support services (i.e., consultation with someone who is thoroughly familiar with the program).

- Quantity, accuracy, and availability of ready-to-use input data for the study area.

*g. Summary.* The following summary steps are suggested as a procedure for selecting an appropriate model for conducting river hydraulics studies.

(1) Define study objectives and required products. Identify project time and personnel availability.

(2) Summarize flow classification, state, regime and type as outlined above and estimate the types of data, amount of data, and quality of data needed to evaluate the types of flow characteristics identified.

(3) Prepare a list of essential data needs in tabular form. Data categories may include:

Hydrologic data (flow records, highwater marks, etc.)
Channel and floodplain geometry data
Sediment data
Geomorphologic and historical data
Other information (e.g., previous studies and reports)

(4) Are the data identified above readily available? Also, are they of the quality and proximity to the study site to be appropriate? Are the data proprietary or public? How up-to-date are they? Develop lists of available and missing data.

(5) Estimate the time and costs associated with the collection of the missing data.

(6) Examine Tables 3-2, 3-3, and 3-4 and compare to the results from the estimation of key hydraulic characteristics. Select the most appropriate methods based on results of this examination.

(7) Consider alternative methods based on results of subsequent studies made such as the reconnaissance study. Continually update and improve methods to meet the specific needs of the study.

# Chapter 4
# Multidimensional Flow Analysis

## 4-1. Introduction

*a. Definitions.* Multidimensional flow analysis is the description and/or prediction of the detailed hydraulic characteristics of a particular flow situation in more than one dimension (direction). "Hydraulic characteristics" refers to the following properties of the flow, discharge, velocity, water surface elevation (depth), boundary shear stress, rate of energy dissipation, and constituent or sediment transport rate. "Particular flow situation" refers to the specific body of water, location therein, physical setting, alternative design configurations, and flows (steady or dynamic) to be studied.

*b. Description.* This type of analysis recognizes velocity and depth variations in either two or three directions. For example, flow patterns in an estuary or at a river confluence may exhibit significant velocities in both the streamwise and transverse directions. A one-dimensional flow model does not explicitly consider these transverse effects. Horizontal, depth-averaged, two-dimensional flow models such as RMA-2 (King 1988, Gee et al. 1990) are used in river hydraulics studies mainly for two purposes: (1) to analyze two-dimensional flow patterns in detail at some area of interest (such as at bridge crossings, the confluence of two channels, flow around islands, etc.) or (2) to analyze the flow behavior on an unbounded alluvial fan or in a wide river valley. Two- and three-dimensional models can be used for both steady and unsteady flow conditions. Sediment transport and water quality analyses can also be done with multidimensional flow models such as TABS-2 (Thomas and McAnally 1985). TABS-2 has primarily been used for simulating the sedimentation processes in reservoirs, estuaries, and complex river channels.

*c. Techniques.* The techniques discussed in this and the following two chapters are strictly applicable only for rigid boundary (bed and banks) situations. Techniques that are used for movable boundary problems (Chapter 7) are extensions of the techniques presented in Chapters 4 through 6. In selecting an appropriate technique, or suite of techniques, the engineer must identify the important physical processes that need to be recognized in the analysis. Resources and data necessary to manage and perform the appropriate level of analysis need to be identified early in the study plan (refer to Chapter 3).

## 4-2. Limitations of One-Dimensional Analysis

Flow in a channel or river is quite often viewed as being one-dimensional in the streamwise direction. This means that the stage (water surface elevation), velocity, and discharge vary only in the streamwise direction. Subdivision of cross sections, however, provides an approximate method of accounting for transverse roughness and velocity distributions. This approach provides a simplified mathematical description of the flow for water surface elevation prediction (see Chapters 5 and 6). More detailed analysis of flow velocities and directions requires representation of the flow physics (conservation of mass and momentum) in two and, sometimes, three dimensions. The engineer should understand the capabilities, limitations, and effort required to perform the various levels of analysis described in this and the following chapters. This information should be used to make an informed decision regarding the technical approach needed to meet the study objectives and to define the resources necessary to manage and perform the study.

## 4-3. Equations of Flow

The principles of mass and momentum conservation are presented below in generalized three-dimensional form. Simplifying assumptions allow the reduction of the equations to two dimensions and to one dimension.

*a. Conservation of momentum.* The conservation of momentum equations in the *x* (horizontal), *y* (horizontal), and *z* (vertical) directions are respectively:

$$\rho\frac{\partial u}{\partial t} + \rho u\frac{\partial u}{\partial x} + \rho v\frac{\partial u}{\partial y} + \rho w\frac{\partial u}{\partial z} - \frac{\partial}{\partial x}(\varepsilon_{xx}\frac{\partial u}{\partial x}) - \frac{\partial}{\partial y}(\varepsilon_{xy}\frac{\partial u}{\partial y}) - \frac{\partial}{\partial z}(\varepsilon_{xz}\frac{\partial u}{\partial z}) - \frac{\partial p}{\partial x} - \tau_x = 0 \tag{4-1}$$

$$\rho\frac{\partial v}{\partial t} + \rho u\frac{\partial v}{\partial x} + \rho v\frac{\partial v}{\partial y} + \rho w\frac{\partial v}{\partial z} - \frac{\partial}{\partial x}(\varepsilon_{yx}\frac{\partial v}{\partial x}) - \frac{\partial}{\partial y}(\varepsilon_{yy}\frac{\partial v}{\partial y}) - \frac{\partial}{\partial z}(\varepsilon_{yz}\frac{\partial v}{\partial z}) - \frac{\partial p}{\partial y} - \tau_y = 0 \tag{4-2}$$

$$\rho\frac{\partial w}{\partial t} + \rho u\frac{\partial w}{\partial x} + \rho v\frac{\partial w}{\partial y} + \rho w\frac{\partial w}{\partial z} - \frac{\partial}{\partial x}(\varepsilon_{zx}\frac{\partial w}{\partial x}) - \frac{\partial}{\partial y}(\varepsilon_{zy}\frac{\partial w}{\partial y}) - \frac{\partial}{\partial z}(\varepsilon_{zz}\frac{\partial w}{\partial z}) - \frac{\partial p}{\partial z} - \rho g - \tau_z = 0 \qquad (4\text{-}3)$$

*b. Conservation of mass.* The conservation of mass equation is:

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \qquad (4\text{-}4)$$

where

$x,y,z$ = the Cartesian coordinate directions.
$u,v,w$ = velocity components in the x,y,z directions, respectively.
$t$ = time.
$g$ = the acceleration due to gravity.
$p$ = pressure.
$\rho$ = fluid density.[1]
$\varepsilon_{xx}$, $\varepsilon_{xy}$, etc. = the turbulent exchange coefficients which describe the diffusion of momentum in the direction of the first subscript to that of the second subscript.
$\tau_x$, $\tau_y$, $\tau_z$ = terms representing the influence of boundary shear stresses.

## 4-4. Significance of Terms

*a. Accelerations.* The terms in these equations represent forces (e.g., the pressure gradient $\partial p/\partial x$), local (temporal) accelerations (e.g., $\partial u/\partial t$), convective accelerations (e.g., $u\partial u/\partial x$), and mass continuity. The momentum equations are derived by application of Newton's Second Law of Motion. The basic assumptions made are that the fluid is incompressible (constant density) and that the effects of turbulent momentum exchange can be simulated with an "eddy viscosity" (Boussinesq assumption). A rigorous derivation of these equations may be found in Rouse (1938) and French (1985).

*b. Forces.* The forces in Equations 4-1 to 4-3 are those of gravity, pressure, boundary friction, and exchange of momentum due to turbulence. Some formulations of these equations may also include forces due to wind, ice, and the earth's rotation. For most riverine situations, wind and the earth's rotation (Coriolis effect) are not important; they may become important for bodies of water with length scales of tens of miles, and may become dominant for large bodies of water such as the Great Lakes. The continuity equation (4-4) represents an accounting of water mass of constant density. Other formulations of these equations, such as used in estuaries, oceans, and lakes may include variable density.

## 4-5. Use of Equations of Flow

*a. General.* Equations 4-1 to 4-4 are applicable to all river and channel flow situations that satisfy the assumptions of constant density and a rigid (or at least slowly changing) boundary. The difficulty lies in solving the equations. The only reliable and routinely used engineering tool for solving the three-dimensional equations at this time (1991) is the physical model. Numerical models (computer programs), however, are routinely and successfully used for solving the two- and one-dimensional simplifications of the above equations. Three-dimensional numerical models are presently under development and undergoing field testing with some applications being reported. A major study of Chesapeake Bay using a three-dimensional numerical model is reported by Kim et al. (1990) and Johnson et al. (1991).

*b. Traditional approaches.* "Traditional" approaches to river hydraulics studies separate continuity, or storage, routing HEC-1, (U.S. Army Corps of Engineers 1990a) to determine the discharge, from the one-dimensional steady flow computations HEC-2, U.S. Army Corps of Engineers 1990b) used to determine water surface elevations. Application of Equations 4-1 to 4-4 achieves the combined result of both routing and water surface elevation computation in a single computation. The "traditional" techniques presented in Chapters 5 and 6 are based on simplifications of, or approximations to, the equations presented above. There are many river analysis problems that can be satisfactorily evaluated with simplified methods. The focus of this chapter, however, is the analysis of more complex hydraulics problems in greater detail and resolution than is available with the traditional techniques.

## 4-6. Two-Dimensional Flow Conditions

*a. General.* For many rivers the width to depth ratio is 20 or more. In these cases, and for many common

[1] In general, density is a function of temperature, salinity, and pressure and is described with an additional "equation of state", see Sverdurp et al. (1942) and Wiegel (1964).

applications, the velocity variations in the vertical are much less important than those in the transverse and streamwise directions. The above equations can be averaged in the vertical (i.e., depth averaged) to yield the two-dimensional equations for flow in the horizontal plane which adequately describe the flow field for most rivers with these characteristics. Two-dimensional flow analysis should be considered for river hydraulics problems where the direction or distribution of flow is of importance, either directly or because it affects variables of interest such as water surface elevation, and cannot be assumed as is required by a one-dimensional analysis. Figure 4-1 depicts a situation where the flow could be adequately modeled by a two-dimensional approach. Figure 4-2 contrasts the one-dimensional approach to the same problem where one must select cross sections perpendicular to the flow direction. While it may be possible to calibrate a one-dimensional model to reproduce the overall energy loss in this flow field, key components of the flow field such as flow separations and recirculation zones would not be reproduced at all by a one-dimensional model.

*b. Specific situations.* Another situation that may require a two-dimensional analysis is that of a bridge with multiple openings crossing a broad, flat, floodplain. In this case the water surface elevation upstream of the bridge may be strongly dependent upon the distribution of flow among the bridge openings. This distribution of flow cannot be directly computed with a one-dimensional approach. Such situations require that the engineer carefully select the level of analysis; physical model, numerical model, or other analytical technique (refer to Chapter 3).

*c. Dynamic simulations.* Multidimensional flow analysis can be either unsteady (dynamic) or steady. Dynamic simulations require substantially more

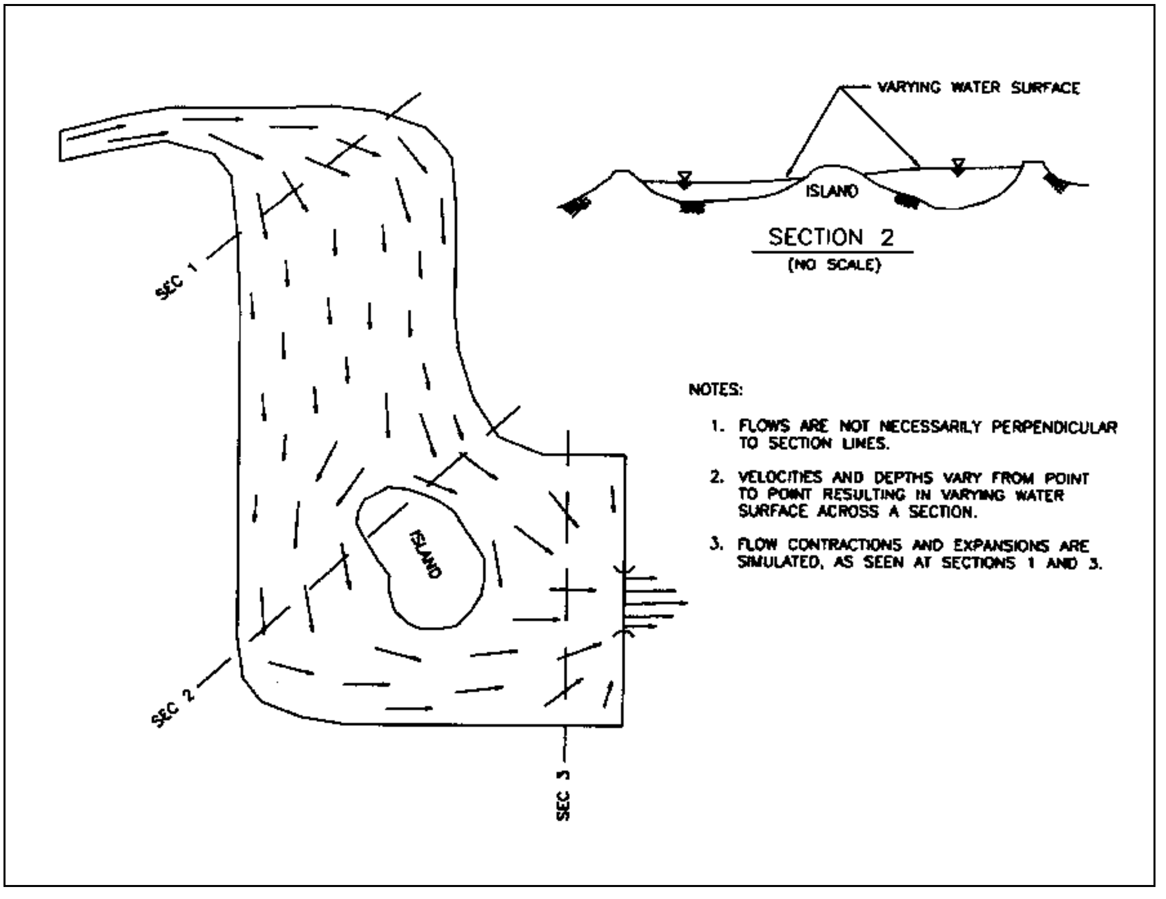


**Figure 4-1. Two-dimensional flow representation in cache creek settling basin**

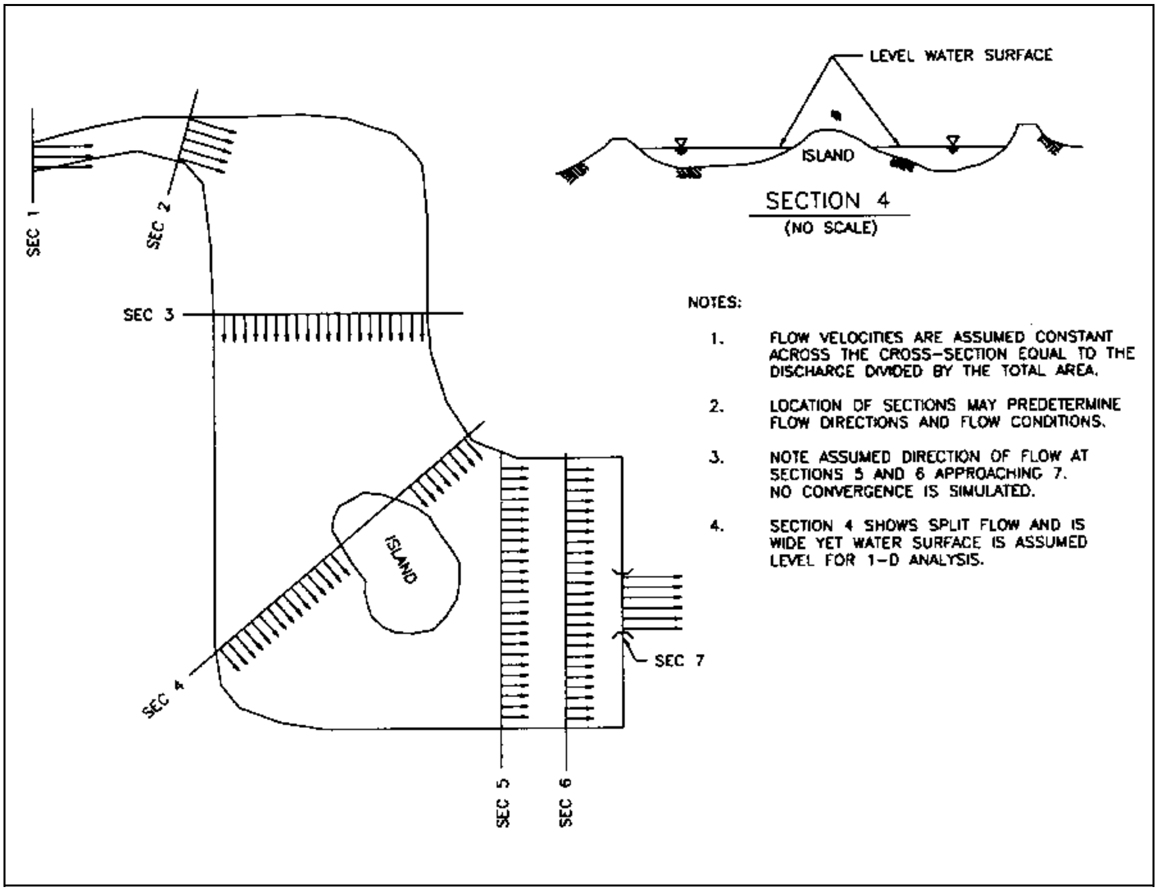


**Figure 4-2. One-dimensional flow limitation in cache creek settling basin**

computational effort than steady state simulations (Gee et al. 1990). Furthermore, the analysis and presentation of results from a dynamic simulation is much more complex than that of a steady flow simulation. Therefore, in designing a multidimensional flow study it is important to decide whether a dynamic analysis is necessary. In most riverine studies, steady flow is adequate; in tidal systems it never is. The alternative design configurations and/or flows to be studied must be carefully selected to maximize study efficiency and ensure that all relevant situations are analyzed. Refer to Appendix C for more detailed information regarding the contents of a work plan for the application of a multidimensional flow model.

## 4-7. Available Computer Programs

*a. Use.* Use of two-dimensional numerical modeling techniques is becoming a routine and accepted engineering practice. Inexperienced analysts should seek guidance and advice from experienced engineers, particularly early in the study, to define data and resources needed for complex model applications. Application of such a sophisticated numerical flow model for a one-time study may best be accomplished with the assistance of a Corps laboratory or outside contractor. Development of in-house expertise for such applications, while requiring significant initial investment of resources in training, may result in future savings if several similar studies are planned. Consideration must be given to model availability (public versus proprietary), applications experience, training and documentation, features, applicability, and required computer resources. Good graphics capabilities, both screen and color hardcopy, are essential to perform efficient and successful applications of multidimensional flow models. Multidimensional flow model applications should be integrated with CADD and/or GIS as appropriate for study needs.

*b. RMA-2.* Computer programs are readily available for conducting two-dimensional river hydraulics analyses in the horizontal plane (Thomas & McAnally 1985, U.S. Department of Transporation 1989). Commonly used in the Corps of Engineers is RMA-2 (King 1988) which is the hydraulics module of the TABS-2 modeling system (Thomas and McAnally 1985). Synopses of these and other programs are presented in HEC (U.S. Army Corps of Engineers 1982b). RMA-2 solves the vertically (i.e., depth) averaged version of equations 4-1 to 4-4; written as shown below.

Momentum equations:

$$h\frac{\partial u}{\partial t} + uh\frac{\partial u}{\partial x} + vh\frac{\partial u}{\partial y} + gh\frac{\partial a}{\partial x} + gh\frac{\partial h}{\partial x} - \frac{h\varepsilon_{xx}}{\rho}\frac{\partial^2 u}{\partial x^2} - \frac{h\varepsilon_{xy}}{\rho}\frac{\partial^2 u}{\partial y^2} + S_{fx} + \tau_x = 0 \qquad (4\text{-}5)$$

$$h\frac{\partial v}{\partial t} + uh\frac{\partial v}{\partial x} + vh\frac{\partial v}{\partial y} + gh\frac{\partial a}{\partial y} + gh\frac{\partial h}{\partial y} - \frac{h\varepsilon_{yx}}{\rho}\frac{\partial^2 v}{\partial x^2} - \frac{h\varepsilon_{yy}}{\rho}\frac{\partial^2 v}{\partial y^2} + S_{fy} + \tau_y = 0 \qquad (4\text{-}6)$$

Continuity equation:

$$\frac{\partial h}{\partial t} + \frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} = 0 \qquad (4\text{-}7)$$

where

$x,y$ = the horizontal coordinate directions.
$u,v$ = velocity components in the x and y directions, respectively.
$t$ = time.
$g$ = the acceleration due to gravity.
$a$ = the bottom elevation.
$h$ = the depth.
$\rho$ = fluid density.
$\varepsilon_{xx}$, $\varepsilon_{xy}$, etc. = the turbulent exchange coefficients which describe the diffusion of momentum in the direction of the first subscript to that of the second subscript.
$S_{fx}$, $S_{fy}$ = terms for the nonlinear Manning or Chezy representation of bottom friction.
$\tau x$, $\tau y$ = terms representing boundary shear stresses other than bottom friction (e.g., wind), these terms also include the Coriolis effect.

## 4-8. Data Requirements

It is useful to think of "data" in three categories: analysis input data, calibration data, and validation or confirmation data. These categories are useful when identifying data requirements for both physical and numerical models.

*a. Analysis input data.* Analysis input data are those items required to operate the model. They consist of a geometric description of the study area (e.g., cross sections in one-dimension, contour maps, or a digital terrain model for two-dimensions), flow to be analyzed (a single discharge for steady flow, or a hydrograph for unsteady flow), other boundary conditions such as stages or rating curves, and various coefficients that approximate the effects of friction and turbulence. Of these, the geometric description of the study area is usually the most time consuming to develop and schematize; it is, however, not necessarily the most important data in terms of simulation accuracy (U.S. Army Corps of Engineers 1986). The density (i.e. resolution) and accuracy required of the flow and geometric data are governed, fundamentally, by the study purpose, not the analysis technique (Cunge et al. 1980).

*b. Calibration data.* Calibration data consist of field observations that are used to evaluate the performance of a model and adjust the coefficients to improve its performance, if necessary. "Performance" is a qualitative, or subjective, measure of the degree to which the model faithfully reproduces the field observations. This measure is applied by the engineer performing the study and documented by means of the reporting process. The complexities of river hydraulics do not allow the setting of objective criteria to measure the accuracy of calibration. Whether the model's performance is acceptable depends on study objectives, sensitivity of study outcomes to model results, and reliability of field data.

(1) The weight given to the performance of a model with regard to different hydraulic variables, such as water surface elevation or velocity, will vary with study objectives, data availability and reliability, and the judgment of the engineer. For example, floodway studies focus on accurate computation of the water surface elevation while constituent transport studies require accurate reproduction of velocity, water discharge, and mixing. Surrogate data should be used with caution. For example, if the study objectives require the prediction of discharge, prototype discharge should be measured for calibration rather than derived from a rating curve.

(2) In the context of two-dimensional modeling for river hydraulics, the study objectives usually require the prediction of velocity or stage. Field measurements of velocity must include the direction as well as the magnitude. Most two-dimensional models used for river hydraulics compute vertically averaged velocities; therefore, the field data must be converted to vertical averages for proper model-prototype comparisons. For most situations, it is adequate to use the average of the velocities measured at 0.2*depth and 0.8*depth (French 1985). Depth must also be obtained at the locations of the velocity measurements. "Depth" alone is of limited value; one should also have the corresponding water surface or bed surface elevation. Similarly, to calibrate a model for stage prediction, one should have field measurements of stage and the variation of stage with time at many locations within the study area. Also, the discharge(s) at the time(s) of those measurements must be known.

*c. Validation data.* Validation data are field observations not used in calibration that are used to provide an independent check on model performance (ASCE 1982). The above considerations for calibration data also apply to validation data.

## 4-9. Data Development and Model Calibration

*a. Geometry.* An accurate geometric description of the flow region is a primary requirement. "Accurate" here means that the key flow controlling and conveying features of the study area are appropriately represented in the field data. The engineer should be aware of the origin and veracity of the field data. Ideally, the area of interest is described by a detailed digital terrain model or contour map of adequate resolution for the study needs. Refer to EM 1110-2-1003 and "Accuracy of Computed Water Surface Profiles" (1986). Most existing model data are, however, in the format of cross sections (HEC-2). Direct use of HEC-2 style data for two-dimensional or one-dimensional unsteady simulations should be tempered by the following considerations: (1) the HEC-2 cross sections may not have been chosen to best represent the direction and distribution of flow, (2) off-channel storage areas (important for dynamic simulations) may have been neglected when surveying the cross sections, and (3) the sections may not be appropriate for the objectives of the present study. Therefore, before using an existing HEC-2 (or other one-dimensional steady flow) data set, thoroughly check the data for conformance with the needs of the present study objectives. The use of cross sections to develop two-dimensional model input requires that the sections be registered (located) on a topographic map or aerial photograph and the contours filled in, usually by hand.

*b. Bottom roughness.* In most two-dimensional riverine situations, bottom roughness can be described in the same fashion as would be used for a "traditional" one-dimensional (HEC-2) analysis (refer to Chapter 6). Due to the ability of the two-dimensional approach to incorporate spatial variation of roughness, aerial photographs or topographic maps can be used to identify regions of uniform roughness, such as clumps of vegetation, changes in bed material or bed forms. As in the one-dimensional approach, the roughness coefficients selected from field inspection (which is essential for successful modeling) will probably need to be modified in the calibration process. Should the calibration process indicate the need for values of coefficients that are outside the range suggested by good engineering judgment, one should closely inspect the geometric data, flow data, boundary condition specifications, and calibration data. Most often it is flawed geometric data, or the manner in which it is interpreted by the engineer and used by the numerical model that is the cause of a poor simulation.

*c. Turbulent exchange coefficients.* Two-dimensional flow models require turbulent exchange coefficients, often called eddy diffusivities, which represent the internal shear forces created by the transfer of momentum between faster and slower regions of flow by means of turbulent mixing. This can actually be observed in most rivers by watching surface boils and eddies move about in the flow. These coefficients reflect, somewhat, the energy losses that are described by the expansion and contraction coefficients in one-dimensional models. The values of these coefficients cannot be directly measured nor observed. Calibrated expansion-contraction coefficients cannot be directly translated into values for the turbulent exchange coefficients. Guidance on selection of values for the turbulent exchange coefficients is provided in the documentation for two-dimensional models (e.g., TABS-2, Thomas and McAnally 1985). These coefficients primarily effect velocity distributions and should be calibrated based on velocity distributions measured in the field. If measurements are not available, information from photographs (both ground and aerial) of the flow or sketches of observed flow patterns can be of use. Some flow situations such as a jet entering a still body of water are momentum dominated. In these cases, the exchange coefficients are very important. Most open river problems are friction dominated, however, and the model results may not be very sensitive to the value selected for the turbulent exchange coefficients. A general approach is to first calibrate the roughness coefficients (Manning's *n* values) to reproduce the energy loss or water surface gradient through the study reach and then adjust the turbulent exchange coefficients to match the observed or expected velocity distribution. The exchange coefficients should be set to the high end of the expected range first, then lowered until the desired velocity pattern is reproduced by the model. In general, the higher the coefficients, the more uniform the velocity distribution; the lower the coefficients, the more readily does flow separation and eddy formation take place. Two-dimensional models (as with one-dimensional models) should be calibrated to steady flow conditions first, if possible, before attempting calibration to an unsteady flow event (Cunge et al. 1980).

*d. Field data.* In addition to thoroughly inspecting the study area, the analyst should be familiar with the manner in which field observations are made, that is, the type of instruments used and the conditions under which the data were obtained. Data reduction techniques may also affect the accuracy and variability of the observations. The analyst should not consider field data to be perfectly accurate nor necessarily representative of field conditions over the complete range of circumstances to be studied. Internal consistency of field data should be checked if at all possible. For example, when using velocity observations for calibration of a two-dimensional model in steady flow conditions, one should calculate the discharge from the velocity and depth measurements and compare it to the discharge obtained from a nearby gage at the same time as the velocity measurements were made.

## 4-10. Example Applications

Most applications of two-dimensional horizontal models to date have been in estuarial environments; some of these applications are presented in "Two-Dimensional Flow Modeling" (U.S. Army Corps of Engineers 1982b), McAnally et al. (1984a, 1984b), and MacArthur et al. (1987). A recent study that evaluated the effects of deepening a ship channel on velocity patterns and shoaling is discussed by Lin and Martin (1989). Computation of velocity distributions in a river downstream from a hydropower project is presented in Gee and Wilcox (1985). Impacts of highway bridge crossings on water surface elevations are discussed in Lee (1980), Tseng (1975), and Heltzel (1988). Effects of dikes on the flow distribution in a river was investigated using TABS-2 by Thomas and Heath (1983). Use of two-dimensional modeling to analyze effects on river stage of a major channel encroachment is presented in Stewart et al. (1985). In this study use of a one-dimensional model did not produce credible results because values of the expansion-contraction coefficients governed the outcome and, as this was a design study, there were no field data for their calibration. Results were much less sensitive to the values of the turbulent exchange coefficients because the major flow patterns and separation areas were calculated directly by the two-dimensional model. It is the effects (energy losses) of these separation areas that the expansion-contraction coefficients attempt to describe. Use of RMA-2 to model flood movement in a large river channel-floodplain system is presented in Gee et al. (1990). This paper also describes the computational resources required to perform such a study. Use of a two-dimensional model to analyze distribution of flow in the St. Lawrence River is documented by Heath (1989).

# Chapter 5
# Unsteady Flow

## 5-1. Introduction

This chapter is presented in two sections. Section I presents guidance on the practical use of unsteady flow modeling and Section II presents some theoretical considerations regarding various routing techniques. Guidance regarding the application of unsteady flow models is presented first because the theoretical information, although important, is of a more general nature.

*Section I*
*Application of Unsteady Flow Models*

## 5-2. Steady versus Unsteady Flow Models

The traditional approach to river modeling has been the use of hydrologic routing to determine discharge and steady flow analysis to compute water surface profiles. This method is a simplification of true river hydraulics, which is more correctly represented by unsteady flow. Nevertheless, the traditional analysis provides adequate answers in many cases. This section identifies when to use unsteady flow analysis.

*a. Steady flow.* Steady flow analysis is defined as a combination of a hydrologic technique to identify the maximum flows at locations of interest in a study reach (termed a "flow profile") and a steady flow analysis to compute the (assumed) associated maximum water surface profile. Steady flow analysis assumes that, although the flow is steady, it can vary in space. In contrast, unsteady flow analysis assumes that flow can change with both time and space. The basics of steady flow analysis were given in Chapter 2; details may be found in Chapter 6.

(1) The typical steady flow analysis determines the maximum water surface profile for a specified flood event. The primary assumptions of this type of analysis are peak stage nearly coincides with peak flow, peak flow can accurately be estimated at all points in the riverine network, and peak stages occur simultaneously over a short reach of channel.

(2) The first assumption allows the flow for a steady state model to be obtained from the peak discharge computed by a hydrologic or probabilistic model. For small bed slopes (say less than 5 feet per mile), or for highly transient flows (such as that from a dam break), peak stage does not coincide with peak flow. This phenomenon, the looped rating curve effect, results from changes in the energy slope. The change in slope can be caused by backwater from a stream junction, as shown in Figure 5-1, or by the dynamics of the flood wave, as depicted in Figure 5-2. Since coincidence of peak stage and flow does not exist in either of these cases, the proper flow to use in a steady flow model is not obvious.

(3) The second assumption concerns the estimation of peak flow in river systems. For a simple dendritic system the flow downstream from a junction is not necessarily equal to the sum of the upstream flow and the tributary flow. Backwater from the concentration of flow at the junction can cause water to be stored in upstream areas, reducing the flow contributions. Figure 5-2 shows the discharge hydrographs on the Sangmon River at the Oakford gage and at the mouth of the Sangmon River 21 miles downstream. The outflow hydrograph is attenuated and delayed by backwater from the Illinois River. Steady state analysis often assumes a simple summation of peak discharges. For steep slopes, once again, the assumption may be appropriate but its merit deteriorates as the gradient decreases.

(4) A more difficult problem is that of flow bifurcation. Figure 5-3 shows a simple stream network that drains a portion of Terrebonne Parish in Louisiana. How can the flow in reach 3 be estimated? Figure 5-1 shows the hydrograph at mile 0.73 in reach 3; note the flow reversal. Hydrologic models and steady state hydraulics cannot predict that division of flow or the flow reversals.

(5) The third assumption allows a steady flow model to be applied to an unsteady state problem. It is assumed that the crest stage at an upstream cross section can be computed by steady flow analysis from the crest stage at the next downstream cross section; hence, it is therefore assumed that the crest stage occurs simultaneously at the two cross sections. Because all flow is unsteady and flood waves advance downstream, this assumption is imprecise. As the stream gradient decreases and/or the rate of change of flow increases, the looped rating curve becomes more pronounced, and the merit of this assumption deteriorates.

(6) The three assumptions are usually justified for simple dendritic systems on slopes greater than about 5 feet per mile. For bifurcated systems and for systems with a small slope, the assumptions are violated and the

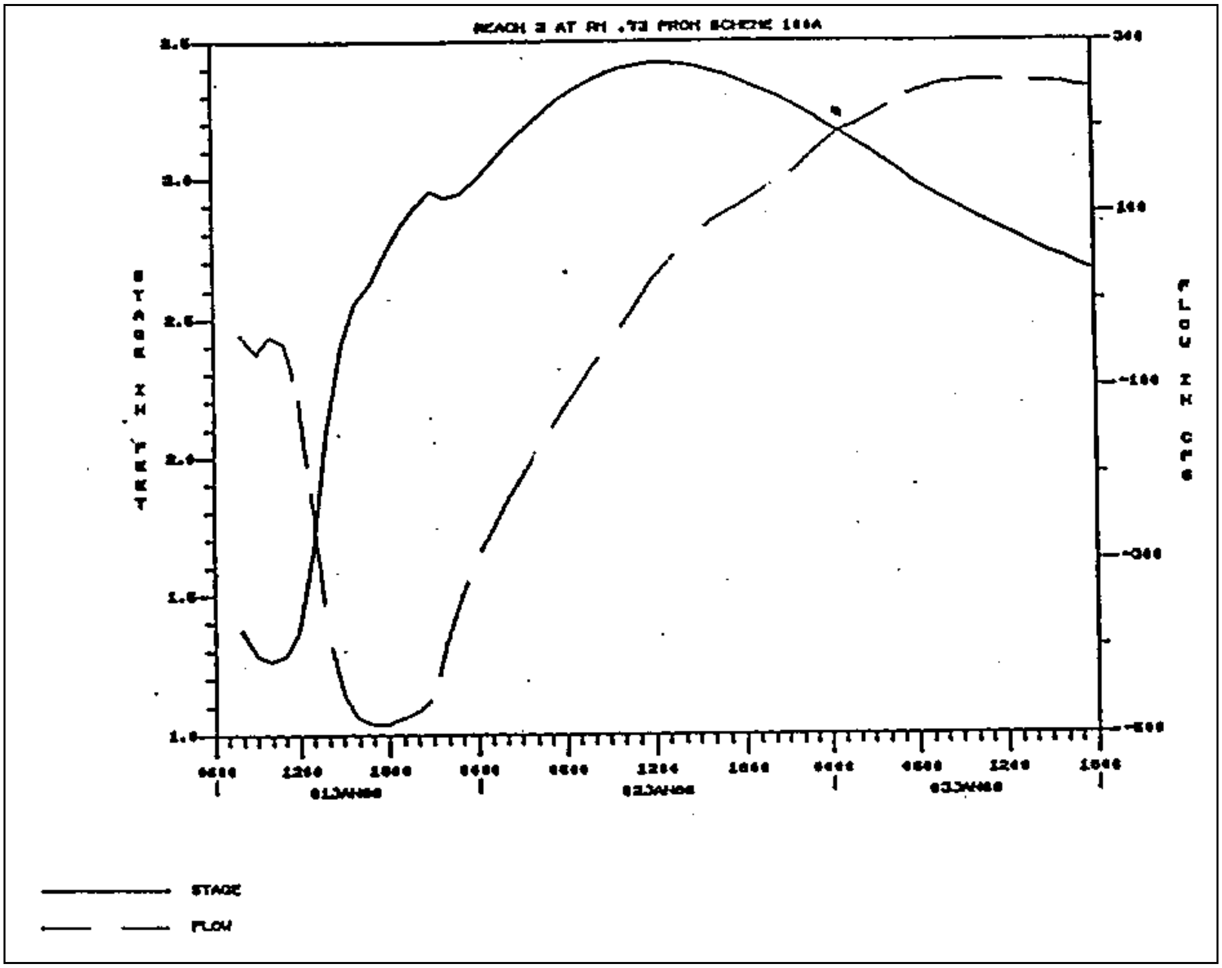


**Figure 5-1. Looped rating curve induced by backwater**

profiles from a steady flow model are suspected. In general, for large rivers and low lying coastal areas, steady flow analysis is not appropriate.

## 5-3. Conditions that Require Unsteady Flow Analysis

Unsteady flow analysis should be used under the following conditions:

*a. Rapid changes in flow and stage.* If the inflow or the stage at a boundary is changing rapidly, the acceleration terms in the momentum equation (see Section 5-12) become important. The leading example is dam break analysis; rapid gate openings and closures are another example. Regardless of bed slope, unsteady flow analysis should be used for all rapidly changing hydrographs. Any information on events of record, high water marks, eyewitness accounts, and so on can be useful in identifying such conditions. Eyewitness accounts of the Johnstown dam-break flood, for example, describe seiching in a major tributary valley. Occupants of floating houses made the trip up and down the valley several times as the currents reversed direction. Only an unsteady flow model with all acceleration terms intact is capable of modeling such an effect on downstream hydrographs and water levels.

*b. Mild channel slope.* Unsteady flow analysis should be used for all streams where the slope is less than 2 feet per mile. On these streams, the loop effect is predominant and peak stage does not coincide with peak flow. Backwater affects the outflow from tributaries and storage or flow dynamics may strongly attenuate flow; thus, the profile of maximum flow may be difficult to

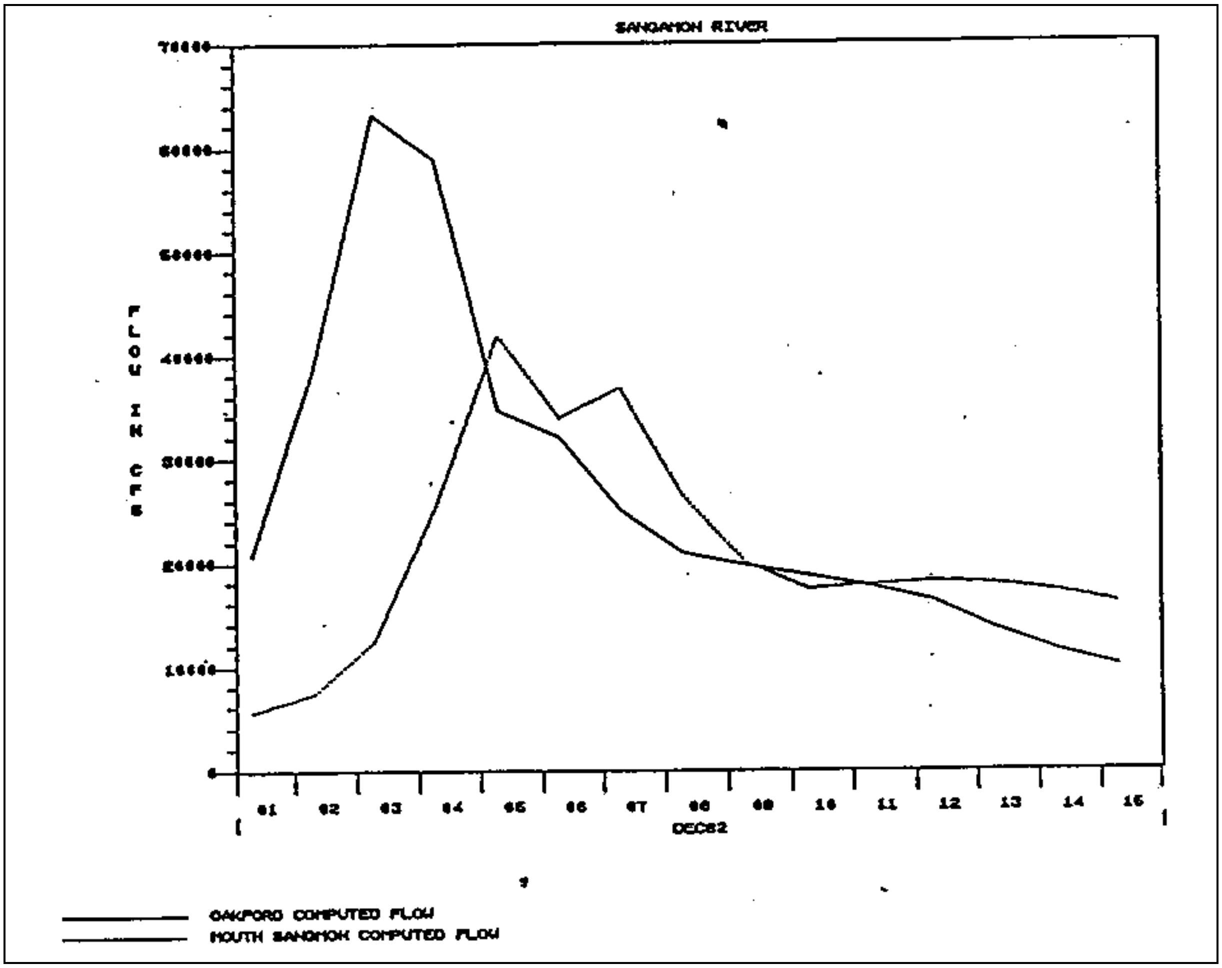


**Figure 5-2. Discharge hydrograph at the oakford gage at the mouth of the Sangmon River**

determine. For bed slopes from 2 to 5 feet per mile, the need for unsteady flow analysis may depend upon the study objectives. Large inflows from tributaries or backwater from a receiving stream may require the application of unsteady flow. Flow reversals may occur under such conditions, rendering hydrologic routing useless. For slopes greater than 5 feet per mile, steady flow analysis is usually adequate if the discharge is correct.

*c. Full networks.* For full networks, where the flow divides and recombines, unsteady flow analysis should always be considered for subcritical flow. Unless the problem is simple, steady flow analysis cannot directly compute the flow distribution. For supercritical flow, contemporary unsteady flow models cannot determine the split of flow. Records of current speeds and directions at different points in a flooded valley and rates of inundation of floodplains help determine whether a one-dimensional approach to a simulation is adequate (see Chapters 4 and 6).

## 5-4. Geometry

The geometry of the reach can be determined from topographic maps, surveyed profiles and cross sections, onsite inspection, and aerial mapping.

*a. Costs.* The influence of errors in reach geometry on predicted stages can be estimated based on regression equations given in "Accuracy of Computed Water Surface Profiles" (U.S. Army Corps of Engineers 1986). Profile errors can also be investigated in a simplified, though representative, reach by modifying its geometry in accord with the possible error and noting the effect on predicted discharges and stages. The costs associated

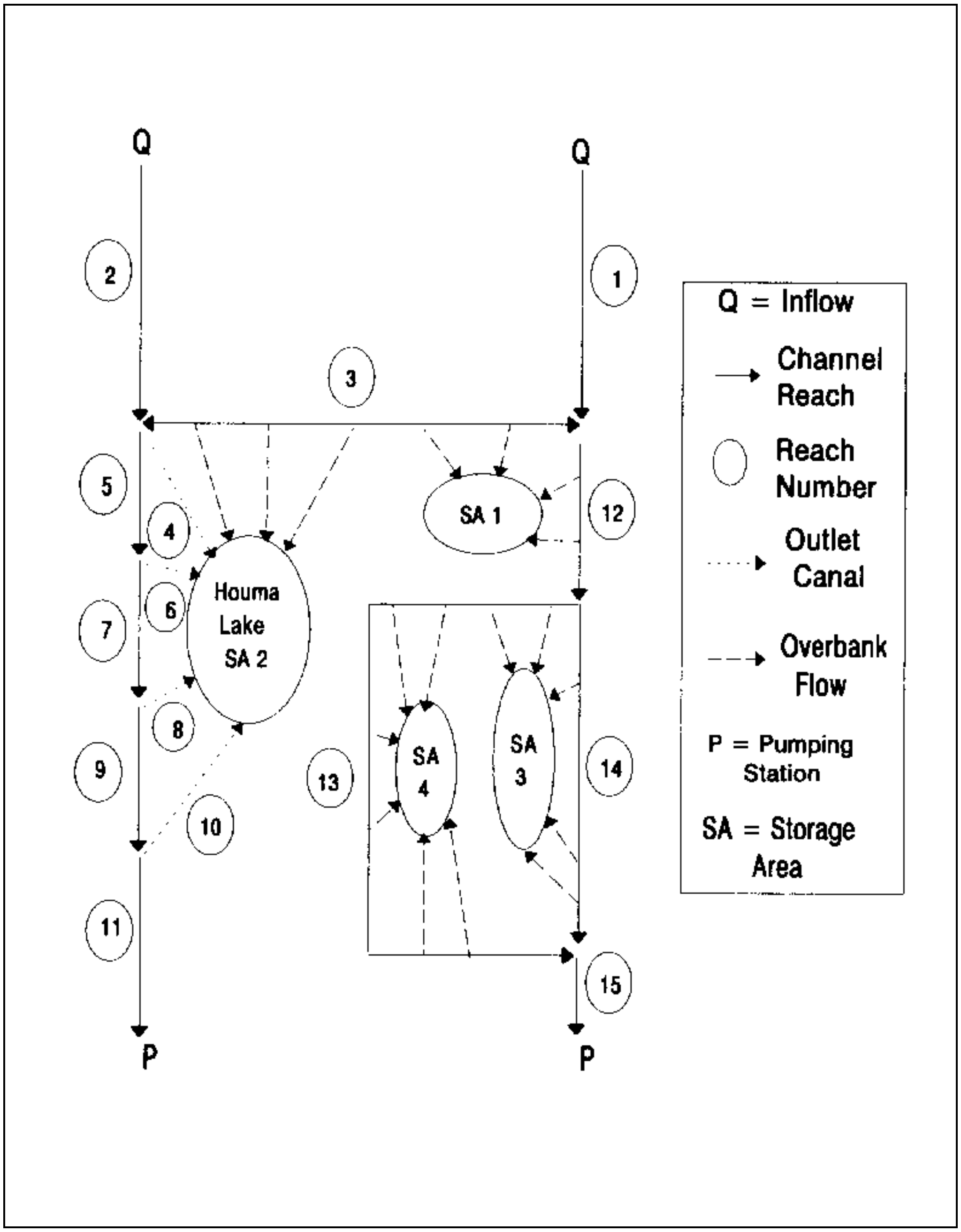


**Figure 5-3. Network of flow system at Terrebonne Parish near Houma, Louisiana**

with surveys of various degrees of accuracy can be estimated from "Accuracy of Computed Water Surface Profiles" (U.S. Army Corps of Engineers 1986, 1989).

*b. Changes.* Visual inspection of a reach must be done to identify the nature of the boundary material, vegetation, and human activities. Alluvium is subject to scour and deposition with possibly major changes in cross section shape accompanying a major flood event. As gross changes in cross section occur within alluvial streams, roughness also changes as dune patterns change during a flood event. Estimated changes in roughness can be applied to a rigid bed model to evaluate the importance of their effect. Prediction of boundary movement lies outside the scope of this Chapter; refer to Chapter 7 and EM 1110-2-4000.

*c. Micro-geometry.* Visual inspection should be used to identify the boundary roughness and other characteristics, such as potential infiltration, of a reach. Infiltration is usually of concern for overland flow; occasionally however, significant water loss (or gain) from a channel will occur in sand, karst, or volcanic geology. Boundary roughness affects the passage of a flood wave. Inspection of the study reach will indicate the nature of the roughness elements: cobbles, boulders, trees, houses, their density and distribution, and variance of roughness with stage and distance down the reach. First approximation values for roughness parameters can be gleaned from past experience with similar roughness elements (Chow 1959, Chapter 5); the drag of trees, and small structures can be estimated from expected velocities, areas of projection normal to the expected flow, and an estimated drag coefficient. Improved values of roughness are obtained by comparing computed and observed flows and stages for events of record.

## 5-5. Controls

*a. Hydraulic controls.* Hydraulic control sections should be sought out because these are natural reach delimiters. At such a section, there is a unique stage-discharge relation (except for the hysteresis induced by unsteady flow), unaffected by flow conditions downstream; hence, it is ideal for a gaging station. It is possible that a control is weak; that is, a rising downstream water level can drown the control section and force its effect upon the subject reach. In that case the reach cannot be studied independently of downstream reaches. This possibility can be investigated with steady flow analyses based on projected flood discharges.

(1) The issue of downstream control is significant to the choice of flood routing method. Influences on water levels within a reach stemming from conditions downstream (tidal levels, or increased levels due to small slope, high roughness, or flow constrictions downstream, for example) preclude application of hydrologic methods. Known water levels (say, tidal) at the downstream end of a reach allow use of hydraulic methods. Otherwise the downstream boundary must be extended until a control (or known level) is encountered.

(2) Downstream from a critical depth control is supercritical flow. If the channel downstream is hydraulically steep and sufficiently long to encompass the reach of interest, supercritical flow will persist all the way down the reach. No independent downstream boundary condition is possible, since downstream depth and discharge are dictated by the flow in the reach. The correct way of modeling such a flow is with an unsteady flow model. If available models cannot deal with supercritical flow, a diffusion model will yield a reasonable result if water surface elevations are not needed and the stream is not extremely flat.

(3) In most cases, the zone of supercritical flow is relatively short, ending either in a plunge into a pool of subcritical flow or joining subcritical flow downstream in a hydraulic jump. In unsteady flow, this jump (called a hydraulic bore) can move about.

*b. Friction control.* A so-called friction control pertains to a section in a nearly uniform reach, sufficiently long to insulate the section from downstream backwater. Then, the stage-discharge relation is governed by a condition of normal depth (near normal in the case of unsteady flow). This type of downstream boundary condition is well suited for all flood routing techniques that recognize downstream boundary influences.

## 5-6. Boundary Conditions

"Boundary conditions" is a mathematical term which specifies the loading for a particular solution to a set of partial differential equations. In more practical terms, boundary conditions for an unsteady flow model are the combination of flow and stage time series, which when applied to the exterior of the model either duplicates an observed event or generates a hypothetical event such as a design flood, or dam break. For an observed event, the accuracy of the boundary conditions affects the quality of the reproduction. In a similar but less detectable manner the reasonableness of the boundary conditions for a hypothetical event (because accuracy can seldom be established) limits the quality of the conclusions. Furthermore, the way that the boundary conditions are applied can control the overall accuracy and consistency of the model.

*a. Upstream boundary conditions.* The upstream boundary condition defines an input to be routed through the system. In most cases this is either a flow or stage hydrograph.

(1) Flow hydrograph. A flow hydrograph is the classic upstream boundary condition where the time varying discharge is routed downstream and the corresponding stages are computed by the model at the upstream boundary and elsewhere. If the flow hydrograph is at a gaging station, the location of the station should be checked. If the station is on a stream with a flat bed slope or with a highly mobile bed, a stage boundary condition may be preferable for reproducing an observed event. However, the flow boundary may be acceptable if the upstream boundary is on a smaller tributary which only makes a minor contribution to the overall system. For this case any error would be lost in the overall system. Be careful when using flows from a slope station as an upstream boundary condition; the values may not be accurate, resulting in an inability to calibrate.

(2) Stage hydrograph. If a stage hydrograph is used as an upstream boundary, the corresponding flow is computed from the conveyance given by the geometric data. Because errors in stage data are less than errors in flow data, the stage hydrograph may have substantial advantages in accuracy over the flow hydrograph. The stage hydrograph is used when a flow station is not available or the quality of flow data is in question. Flow computed from a stage boundary must always be verified against reliable flow measurements, otherwise substantial error in flow can result. If no flow measurements are available, the stage hydrograph should only be used when absolutely necessary and then with caution. Figure 5-4 shows the reproduction of flow measurements at Hickman from routing Cairo stages down the Mississippi River. Figure 5-5 shows the reproduction of stage at Memphis 200 miles downstream.

*b. Downstream boundary condition.* For subcritical flow, the downstream boundary condition introduces the effect of backwater into the model. Four types of downstream boundary conditions are stage hydrograph, flow hydrograph, rating curve, and Manning's equation.

(1) Stage hydrograph. The classic downstream boundary is the stage hydrograph. The corresponding flow is calculated by the model. Because the stage hydrograph is observed, and therefore presumed accurate, the downstream end of a study reach can be located at a gage.

(2) Flow hydrograph. The flow hydrograph is a special purpose downstream boundary condition which is generally used to simulate a reservoir outflow or a pumping station if accurate outflow is known. For the flow hydrograph, the model calculates the corresponding stages. The time series of computed stages is based on an initial stage and will change with a differing initial stage. The flow hydrograph must be used with great care because the flow is leaving the system and negative depths may be computed, in particular at pumping stations.

(3) Rating curve. A single valued rating curve describes a monotonic relationship between stage and flow. The rating curve is accurate and useful for describing a boundary condition at a free overfall, such as a spillway or at a falls, or at a pump station whose performance is defined by a schedule. But the single valued rating curve is often a poor downstream boundary condition for a free flowing stream. Natural rivers display a looped rating curve; use of a single valued rating curve, however, forces a monotonic relationship which erroneously reflects waves upstream. For this reason, the rating curve must be located well downstream of the reach of interest in a free flowing stream to prevent errors from propagating upstream into the area of interest. This lack of sensitivity should be confirmed by sensitivity tests.

(4) Manning's equation. Manning's equation can be used as a downstream boundary condition for a free flowing stream when no other boundary condition is available. The model computes both stage and flow with the stage being a function of the friction slope. Two methods prevail for determining the friction slope. Fread (1978, 1988) in DWOPER and DAMBRK assumes that the friction slope is equal to the water surface slope.

UNET (U.S. Army Corps of Engineers 1991b) uses the friction slope at the last cross section. These two assumptions, which produce slightly different results, are

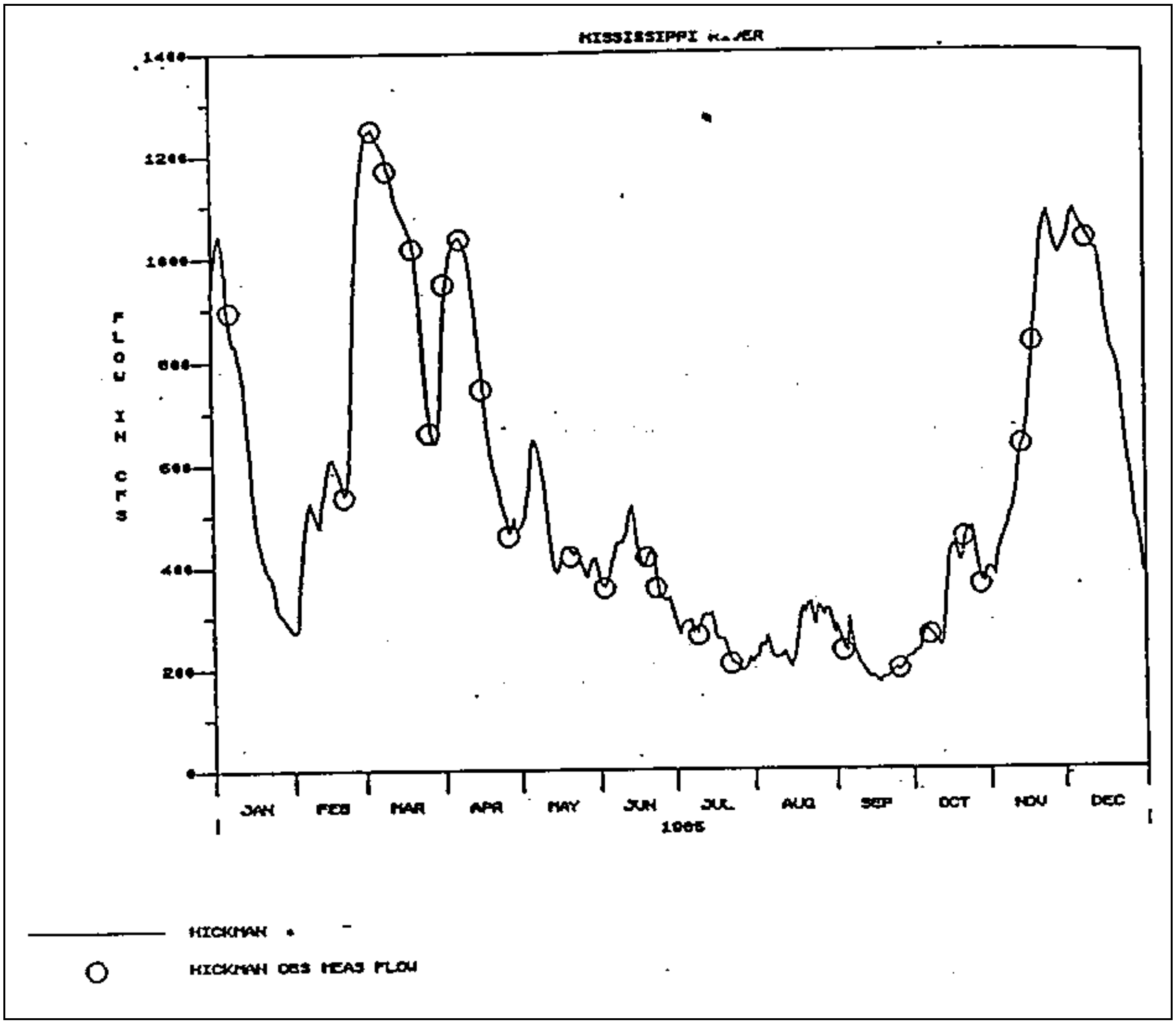


**Figure 5-4. Computed versus observed flow at Hickman**

both reasonable. Because of the variable friction slope, Manning's equation does display the looped rating curve; but the boundary condition still should be placed well downstream of the area of interest. Any model which uses Manning's equation as the downstream boundary condition should be tested for sensitivity to confirm that its use at the boundary has no affect on the area of interest.

*c. Lateral inflow.* Lateral inflow (or outflow for a diversion) data also constitute a boundary condition. Unlike upstream and downstream boundary conditions which translate into an independent equation, lateral inflow ($q_L$) augments the equations of continuity and momentum (see Equations 5-2 and 5-3). Lateral inflow can come from gaged and ungaged areas, and can be located at a point and/or uniformly distributed along the length of the valley.

(1) In any river system a part of the drainage will not be gaged. To provide an accurate and consistent simulation, the modeler must estimate the inflow from those ungaged areas. Along the Illinois River, for example, there is 2,579 square miles of ungaged drainage between the Marseilles and Kingston Mines gages, which is about 52 percent of the total gaged area. Figure 5-6 shows a simulation result at Kingston Mines without the ungaged drainage. The omission of the ungaged drainage produced a uniform error of about 1 foot in the simulated

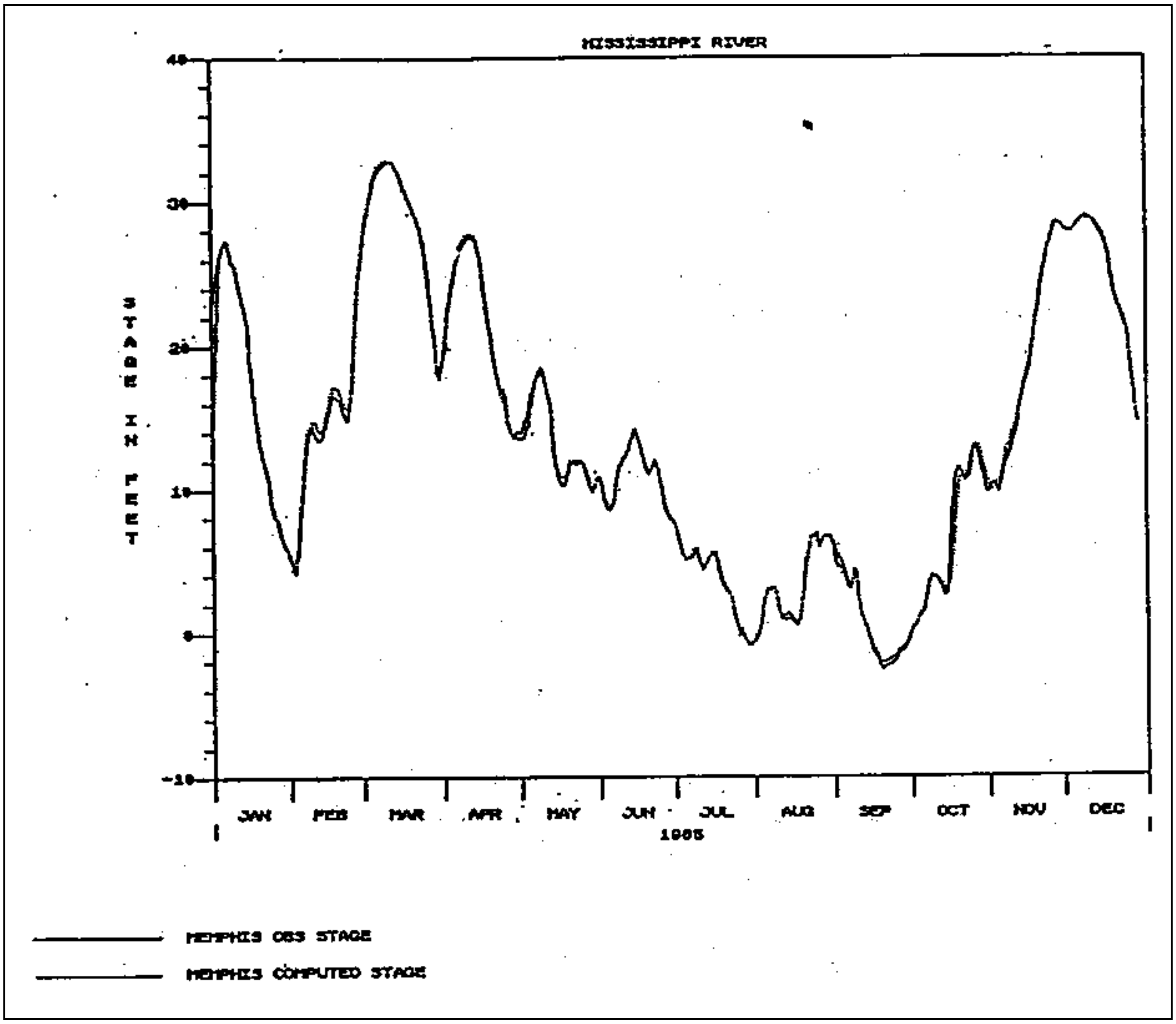


**Figure 5-5. Reproduction of Stage at Memphis from Stages Routed from Cairo, 220 Miles Upstream**

stage hydrograph. This difference could have been corrected by adjusting the *n* values, but the error would have become apparent as an inconsistency when verifying against other events. Figure 5-7 shows the correct simulation which includes the ungaged inflow.

(2) The estimation of ungaged inflow is difficult because of the wide variation in spatial rainfall patterns. Two methods are proposed: estimating runoff using drainage area ratios applied to gaged watersheds in the vicinity and use of a rainfall-runoff model. Drainage area ratios work well for large events when the rainfall is relatively uniformly distributed spatially. For smaller events, which cause small peaks in low flow, the method is less appropriate. A hydrologic model is preferable, but it may be an additional study step to develop and maintain, and requires precipitation data. Small, often unnamed, tributaries may be lumped together and entered uniformly as a single hydrograph which is distributed along a portion of the stream. Generally, the distribution is according to floodplain distance. Uniform lateral inflow is for the convenience of the modeler.

(3) Lateral inflow from a gaged tributary or from a large ungaged tributary is usually entered at a point. For streams with a flat bed slope a tributary inflow causes a disruption in the stage profile, as shown in Figure 5-8 by the correspondence between flow and stage discontinuities. Locating point inflows, even for ungaged areas,

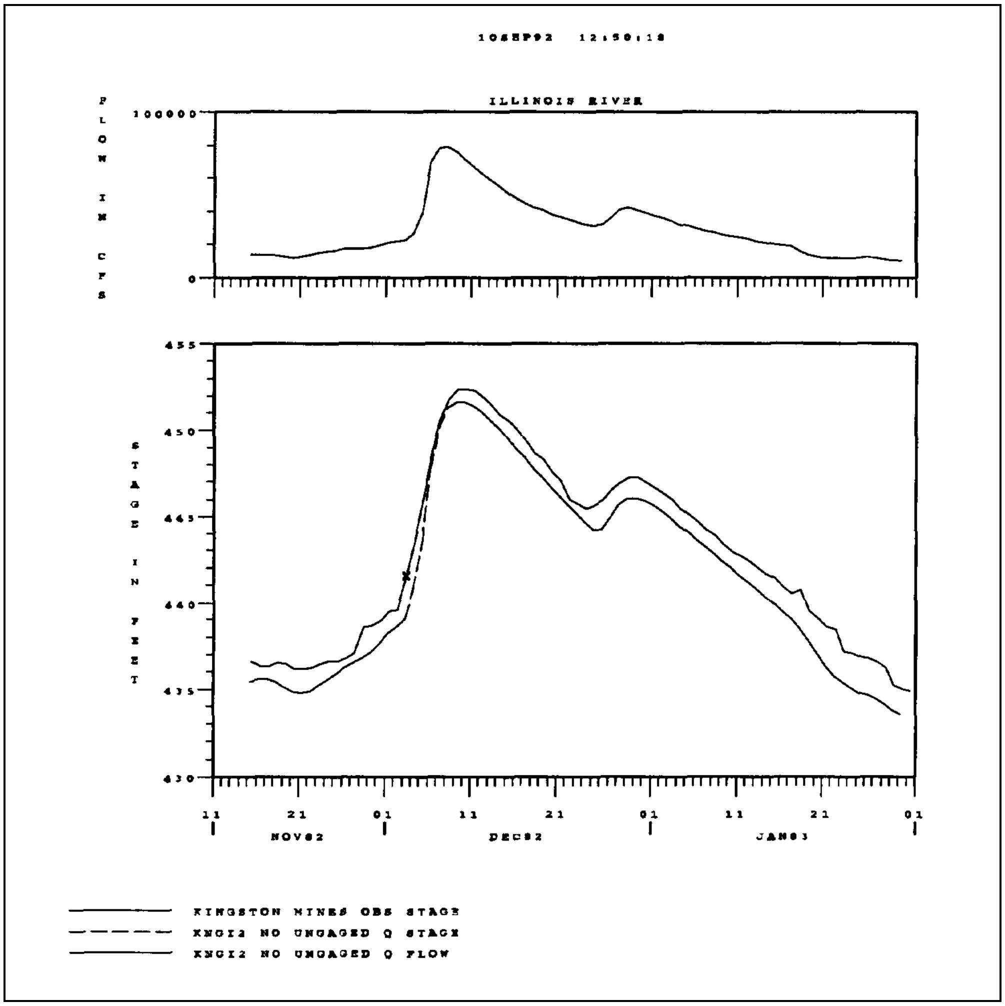


**Figure 5-6. Simulation of the Illinois River at Kingston Mines without 2,579 square miles of ungaged drainage**

may be a determining factor in the accuracy of the model. For the Illinois River, unsatisfactory results were produced if inflows from greater than 100 square miles were not entered at a point.

## 5-7. Steps to Follow in Modeling a River System

The following is a sequence of steps to follow when modeling a river system using unsteady flow. In subsequent sections, some of these topics will be expanded.

*a. Prepare schematic diagram.* The basic schematic diagram shows the layout of the river system and the principal tributaries for which gaged flow data are available. It is best to model every tributary for which cross-sectional data are available since the degree of detail determines the accuracy and consistency of the simulation. Also, tributaries can be modeled at modest additional cost in computer time and engineer effort. The scope of the model should be large enough so that errors in the downstream boundary condition do not affect results at the locations of interest. An example schematic diagram for the Red River of the North is shown in Figure 5-9 (U.S. Army Corps of Engineers 1990c).

*b. Collect cross-sectional data.* Collect all the cross-sectional data available on the main stem and tributaries. If data are old, or suspect for any reason, new data may be required. Usually cross section data are unavailable on all but the largest tributaries, thus limiting

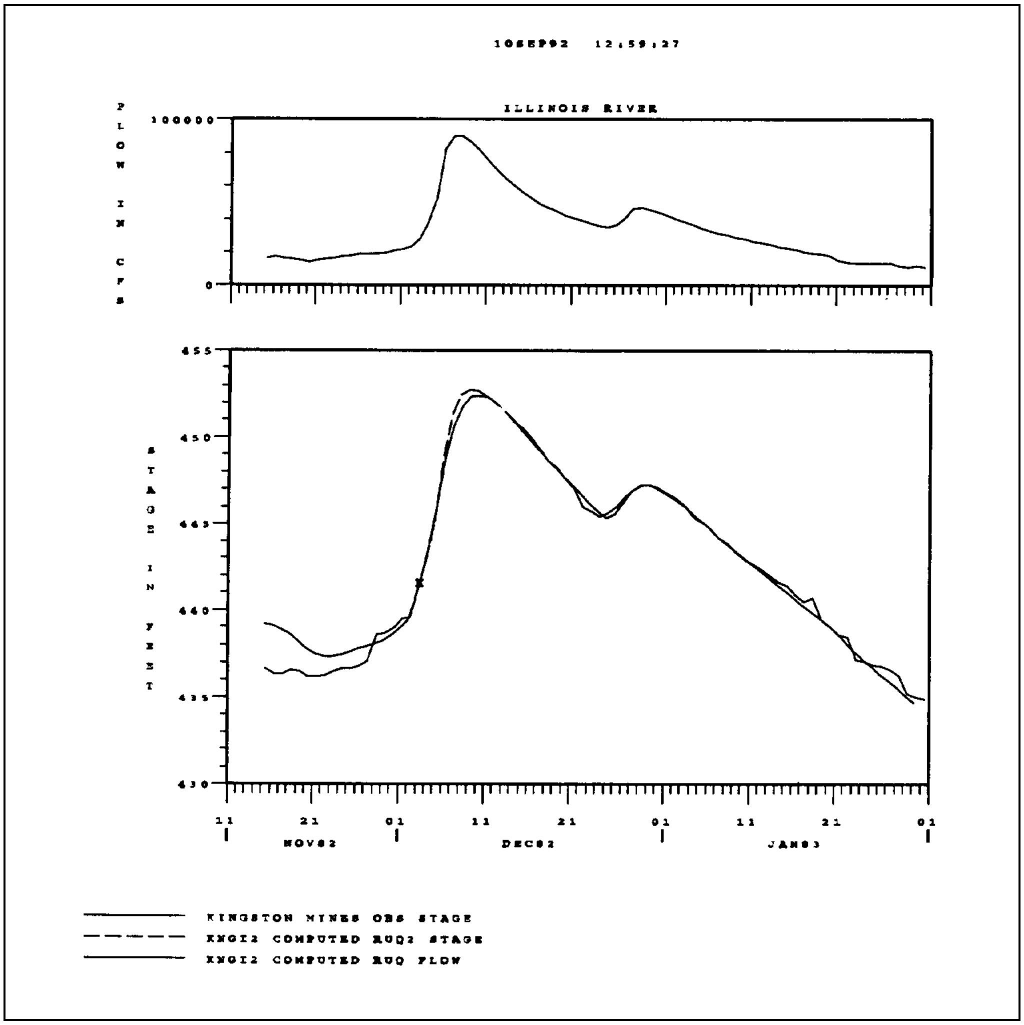


**Figure 5-7. Simulation of the Illinois River at Kingston Mines including 2,579 square miles of ungaged drainage**

the scope of the model. Study funds may limit the number of new surveys. If a major tributary has no surveyed cross sections, consider approximating the channel cross section and obtaining overbank information from USGS quadrangle maps. Remember, accuracy and consistency depend on the degree of detail. Details of cross section positioning are presented in Appendix D.

*c. Collect stream gage data.* Collect flow and stage data for the main stem and all tributaries. It is recommended that a data base such as HEC-DSS (U.S. Army Corps of Engineers 1990d) be used to organize observed data and maintain, display, and analyze computed results.

*d. Develop gaging table.* Develop a table showing all stream gaging locations from upstream to downstream, all major tributaries with gages, all major tributaries without gages, and reaches with uniform lateral inflow. For an unsteady flow simulation to be successful, every square mile of drainage must contribute inflow to the model. The gaging table locates the ungaged drainage and identifies the source from which ungaged inflow will be estimated. Table 5-1 is such a table for the Lower Mississippi River.

*e. Revise schematic diagram.* Revise the diagram by identifying all the reaches to be modeled, the locations of the gages, and all inflow points. To some extent, the gaging table and the schematic diagram are redundant, but the graphical display in the diagram helps assure an accurate definition of the system.

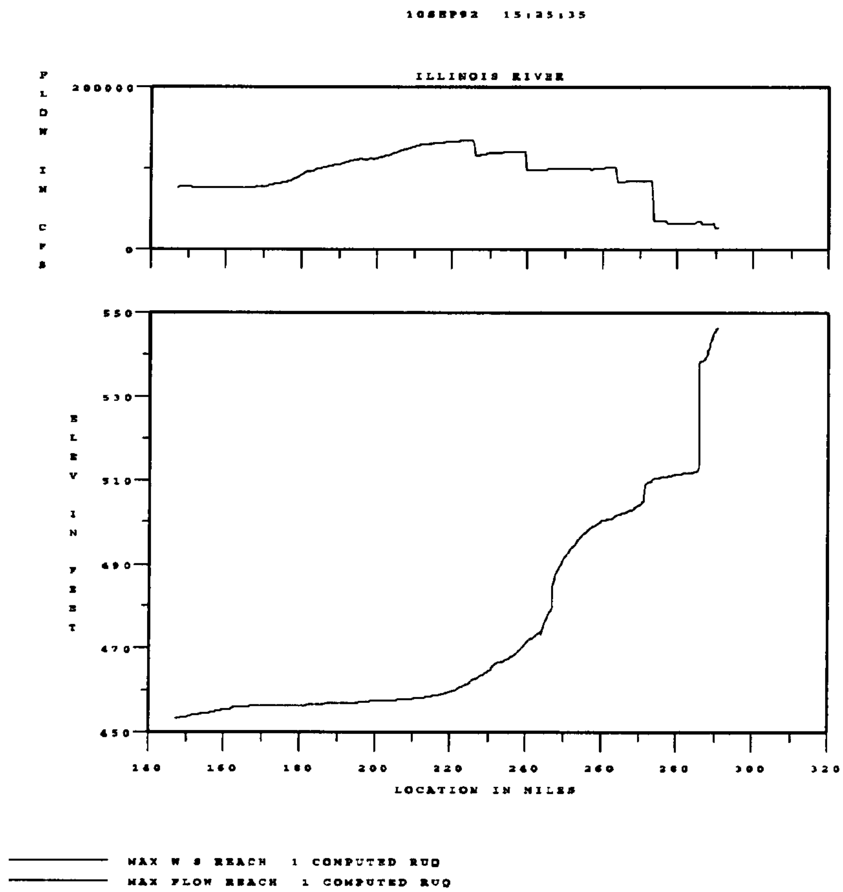


**Figure 5-8. Disruption of the stage profile of the Illinois River by inflow from the Fox River**

*f. Assemble cross section file.* On the basis of the schematic diagram, prepare the geometric data file. See Appendix D.

*g. Identify a calibration event.* Choose a time period that includes one of the largest events of record. The period should also include low flow and should contain the maximum amount of stage data.

*h. Assemble boundary condition file.* From the gaging table and the schematic diagram, assemble the boundary condition file locating all point and uniform lateral inflows in their proper locations.

*i. Calibration.* Calibrate the data to reproduce the calibration event.

*j. Verification.* Verify the simulation using other periods and events in the record. Minor adjustments to the parameters are acceptable, but no major changes should be needed. If the reproduction is inadequate, attempt to identify why.

## 5-8. Accuracy of Observed Data

All observed data are subject to measurement error. Both the operation and calibration of an unsteady flow model are based primarily on flow and stage data from gaging stations. Some stations have better records than others. It is the management of the error which results in the quality and consistency of the model. Consistency is the ability to reproduce multiple events with a single calibrated data set.

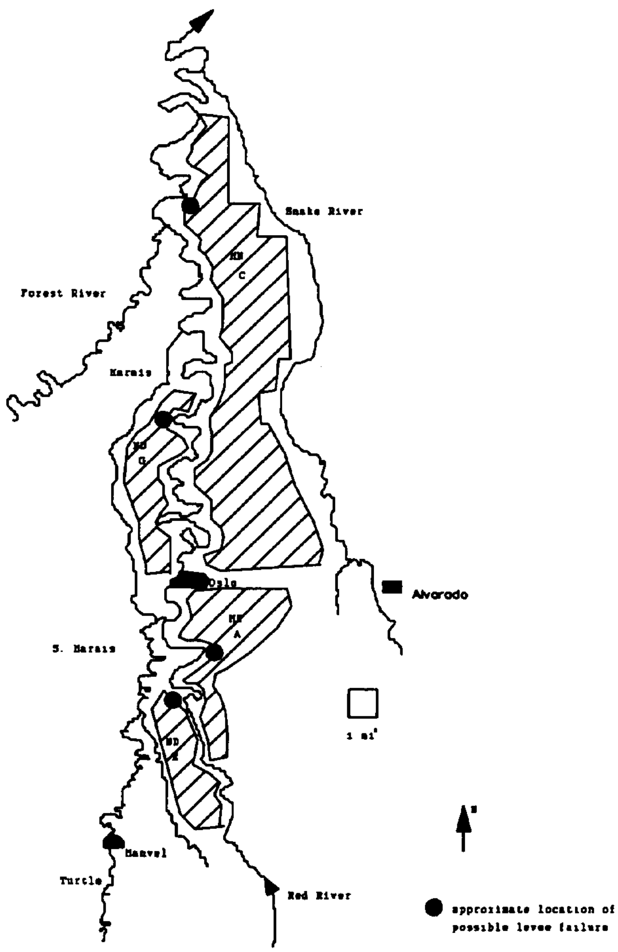


Figure 5-9. Schematic diagram for the Red River of the North

**Table 5-1**
**Gaging Table for the Lower Mississippi River**

| MISSISSIPPI RIVER | | | | | TRIBUTARY DATA | | | |
|---|---|---|---|---|---|---|---|---|
| Mississippi River Gage | River Mile | Drainage Area (Sq. Mi.) | Gage Zero (Ft NGVD) | Ungaged Drainage (Sq. Mi.) | Tributary | Gaging Station | Drainage Area (Sq. Mi.) | River Mile |
| Chester, IL | 109.9 | 708,563 | 341.05 | | | | | |
| Thebes, IL | 43.7 | 713,200 | 300.00 | 4637 | | | | |
| Birds Point, MO | 2.0 | 713,397 | | 197 | | | | |
| | | | | | Ohio River | L&D 52 (TW) | 203000 | |
| | | | | | Ohio River | Cairo | 203040 | 953.8 |
| Wickliffe, KY | 951.5 | 917,400 | 269.12 | 963 | | | | |
| Columbus, KY | 937.2 | 917,900 | 266.38 | 500 | | | | |
| Hickman, KY | 922.0 | 918,500 | 264.73 | 600 | | | | |
| New Madrid, MO | 889.0 | 919,200 | 255.48 | 700 | | | | |
| Caruthersville, | 846.4 | 919,400 | 235.49 | 200 | | | | |
| Cottonwood Pt. MO | 832.7 | 919,500 | 230.18 | 100 | | | | |
| | | | | | S. Fk. Deer R. | Halls, TN | 1014 | |
| | | | | | N. Fk. Deer R. | Dyersburg, TN | 939 | |
| | | | | | Obion River | Bogata, TN | 2033 | |
| | | | | | Obion River | | 3986 | |
| Gage 158 | 819.1 | 924000 | 218.33 | 514 | | | | |
| Osceola, AR | 783.5 | | | | | | | |
| Fulton, TN | 778.2 | 924,300 | 208.61 | 300 | | | | |
| | | | | | Hatchie R. | Rialto, TN | 2308 | 773.3 |
| Richardson, TN | 769 | | | | | | | |
| | | | | | Wolf River | Raleigh, TN | 770 | 738.6 |
| Memphis, TN | 734.7 | 928,700 | 183.91 | 1322 | | | | |
| Star Landing, MS | 707.4 | | | | | | | |
| Mhoon Landing, MS | 687.5 | 929,200 | 161.22 | | | | | |
| | | | | | St. Francis Bay | Riverfront, AR | 5141 | 672.4 |
| | | | | | St. Francis R. | Parkin, AR | 900 | |
| Helena, AR | 663.1 | 937,700 | 141.7 | 2459 | | | | |
| Fair Landing, AR | 632.5 | 937,800 | 132.2 | | | | | |
| | | | | | White River | Clarendon, AR | 25497 | 599 |
| Nr Rosedale, MS | 592.1 | 965,800 | 108.73 | | | | | |
| | | | | | Arkansas R. | Pine Bluff, AR | 138000 | 581.4 |
| Arkansas City, AR | 554.1 | 1,104,360 | 96.66 | 560 | | | | |
| Greenville, MS | 531.3 | 1,104,460 | 74.92 | | | | | |
| Lake Prov, LA | 487.2 | 1,104,560 | 69.71 | | | | | |
| | | | | | Yazoo River | Yazoo City, MS | 8900 | |
| Vicksburg, MS | 435.7 | 1,118,160 | 46.23 | | | | | |
| St. Joseph, LA | 396.4 | 1,122,660 | 33.12 | | | | | |
| Natchez, MS | 363.3 | 1,123,160 | 17.28 | | | | | |
| Knox Landing, LA | 313.7 | 1,124,700 | 0.00 | | | | | |
| Red R. Landing, | 302.4 | 1,125,000 | 0.00 | | | | | |
| Bayou Sara, LA | 265.4 | 1,125,400 | 0.00 | | | | | |
| Baton Rouge, LA | 228.4 | 1,125,810 | 0.00 | | | | | |
| Plaquemine, LA | 208.8 | 1,125,830 | 0.00 | | | | | |
| Donaldsonville, LA | 175.4 | 1,125,860 | 0.00 | | | | | |
| Reserve, LA | 138.7 | 1,125,880 | 0.00 | | | | | |
| Bonnet Carre, LA | 128.0 | 1,125,890 | 0.00 | | | | | |
| New Orleans, LA | 102.8 | 1,125,910 | 0.00 | | | | | |
| Chalmette, LA | 91.0 | 1,125,920 | 0.00 | | | | | |
| West Pointe, LA | 48.7 | 1,125,940 | 0.00 | | | | | |
| Empire, LA | 29.5 | 1,125,960 | 0.00 | | | | | |
| Fort Jackson, LA | 18.6 | 1,125,965 | 0.00 | | | | | |
| Head of Passes, LA | -.6 | 1,125,970 | 0.00 | | | | | |

*a. Stage data.* Stage data are the most accurate type of hydrologic data. Stage measurement is accurate to within the amplitude of wind induced gravity waves and the consistency of the recording device. Experience has shown that gravity waves are typically about ± 0.1 foot in magnitude; see Figure 5-10. Traditional recording devices, e.g., strip chart recorders and paper tapes, which were predominant until the early 80's, tended to lose their accuracy with time. Each month, when the gage reader changed the tape, the automatic and the manual gage readings were recorded. Usually the difference was a couple of tenths of a foot although, occasionally, big discrepancies were found. The recorded readings were typically then adjusted by a linear relationship with time to match the manual reading under the assumption that the error increased gradually with time. The validity of this assumption may be questionable. These errors, which may be hidden, have bearing on how well the model seems to match observed data. Another problem is that gages sometimes lose their datum. Figure 5-11 shows a comparison of the Des Plaines River stages at Lockport with those at Brandon Road Pool, which is downstream. For 1974, Brandon Road is higher than Lockport; hence, the Des Plaines River appears to be flowing backwards. Which gage is correct?

(1) Newer gages have electronic recorders and transmit data via satellite. Still, the gages are subject to the similar losses of accuracy with time. Also, satellite transmissions are subject to large errors which appear as spikes in the time series. These spikes are easy to discern, but if they are input to a simulation they are disastrous.

(2) Finally, point observations, say the 07:00 reading, are often read from the hourly satellite time series. Since the data may be oscillating (Figure 5-10) is one point representative of the overall time series?

*b. Flow data.* Flow is usually a derived, not a measured quantity. Periodic flow measurements, using velocity meters, are initially used to define a rating curve

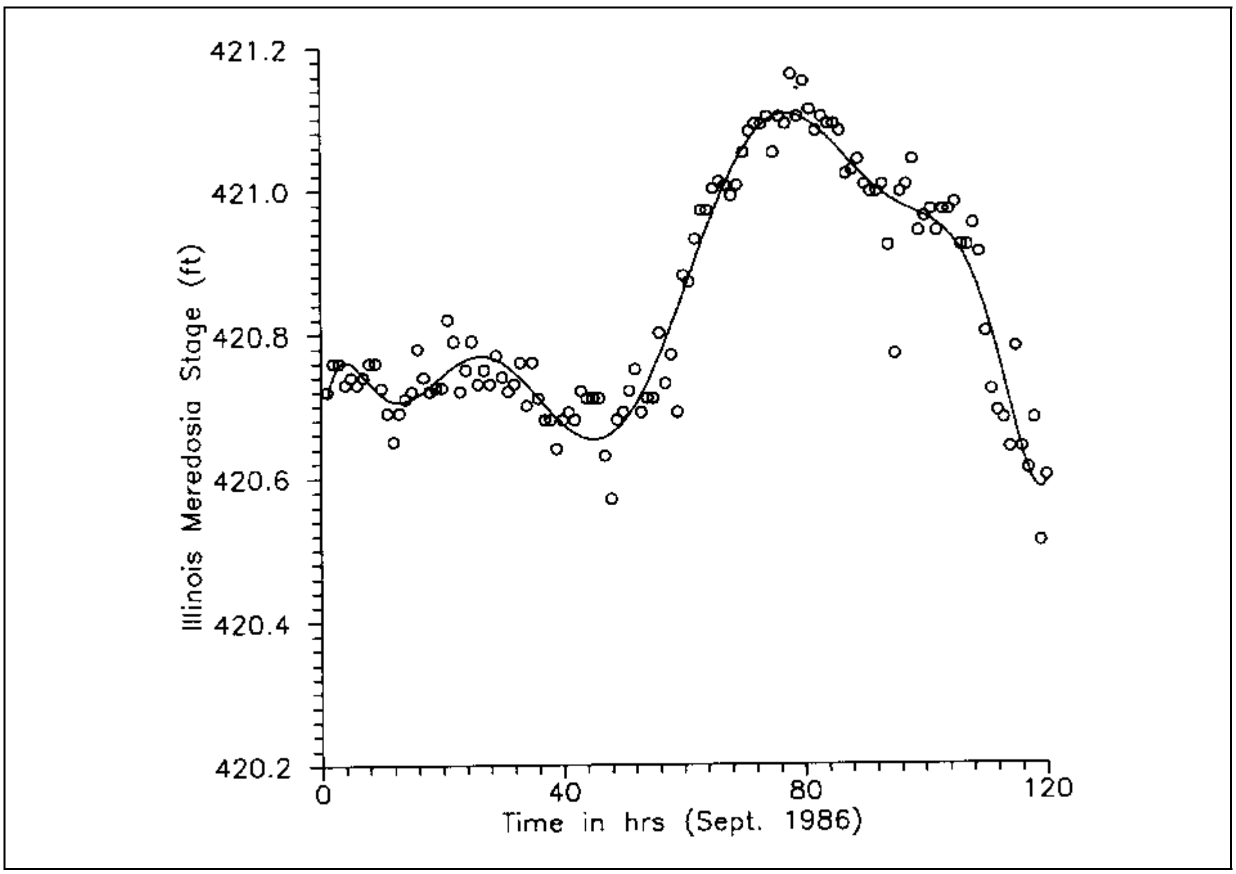


**Figure 5-10. Oscillation of the 1-hour time series from a satellite for the Illinois River at Meredosia**

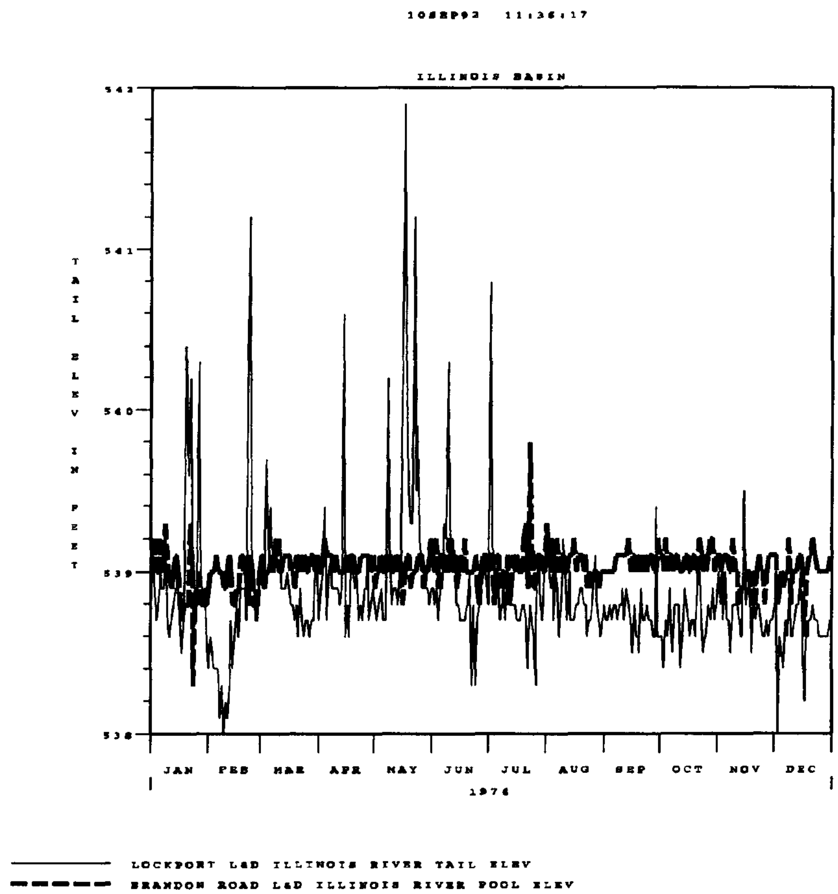


**Figure 5-11. Stage hydrographs for the Illinois Waterway at Lockport Tailwater and Brandon Road Pool**

and then to define shifts (seasonal, systematic, and random) from the rating curve. The "shifted" rating curve is then used to routinely derive discharge from stage with the discrete flow measurements being the only solid data.

(1) The USGS defines an "excellent" gaging station as having 95 percent of the daily discharges within ± 5 percent of the true value. The departure of the measurement from the rating curve is composed of the error in the measurement and the true shift. The shift is manually determined by attempting to isolate the error. The records at upland stations where the bed slope is large are usually good. On the other hand, the records on large rivers, where the bed slope is small and the dynamics are large, are suspect.

(2) The ability to adequately determine the rating curve shift depends on the frequency of discharge measurement. Long term trends of aggradation and degradation are adequately defined by even an infrequent measurement cycle. However, unless several measurements are taken during a flood event, it is unlikely that the loop or a seasonal shift will be adequately defined, resulting in an error. When modeling a river system, if a gaging station is used as an upstream boundary, the error results in an inconsistency in calibration between events which cannot be reconciled. On the Middle Mississippi River a base flow error of ± 5 percent results in a model inconsistency of ± 1 foot. If the lack of definition of the loop is added to the base error, sizable inconsistency can be explained. Slope stations are gaging stations which

are influenced by backwater. At these stations the rating curve is modified not only by the shift but also by a slope correction which is computed from the observed fall to a downstream gage. Discharge records at a slope station are seldom very good and should be used as boundary conditions with caution.

(3) There are gaging stations whose records are not very reliable. These are usually on streams with a flat bed slope or a mobile boundary. At these locations, only the actual flow measurements can be used with confidence.

## 5-9. Calibration and Verification

When a model is calibrated, the parameters which control the model's performance, primarily Manning's *n* and reach storage, are determined. The key to a successful calibration is to identify the true values of the parameters which control the system and not to use values that compensate for shortcomings in the geometry and/or the boundary conditions. Because unsteady flow models reproduce the entire range of flows, they should be calibrated to reproduce both low and high flows.

*a. Manning's n.* In the unsteady flow models used in the United States, the friction slope is generally modeled using Manning's equation. Manning's *n* value relates the roughness of the stream boundary to the friction force exerted on the system. For most problems, an initial estimate of Manning's *n* (it is only an educated guess) is used at the start of the calibration. The initial values are then adjusted to match observed stage data. When no observed stage data exists, the estimated values take on a greater importance since they are assumed to be representative of the system. See Appendix D for detailed information on selecting *n* values.

*b. Calibration.* Calibration of an unsteady flow model is a four step process. In the first step the *n* values are adjusted to reproduce the maximum stages of an event. The storage in the cross sections is then adjusted, if necessary, to improve timing and attenuation. In the third step, the flow versus Manning's *n* relationship is adjusted to reproduce both high and low flow event stages. Finally, the model is fine tuned to reproduce a longer period which should include the initial calibration event.

(1) The initial calibration event should be one of the larger events which are available in the time series. The purpose of this phase is to adjust the initial *n* values to match the crest of the event at all stations in the model. Figure 5-12 shows the hydrographs for the Illinois River at Havana after the initial calibration. Note that, although the crest stage is approximately correct, the timing of the hydrograph and the reproduction of low flow are deficient.

(2) Total storage as defined by river cross sections is almost always deficient. In natural rivers, the timing of the hydrograph is determined by storage and the dynamics of the flood wave. Timing can be adjusted by modifying storage, friction, and distribution of lateral inflows. If the timing cannot be calibrated by reasonable adjustment of these factors, then there is some other problem, most likely an error in the cross sections. For the Illinois River, which is confined by levees in the reach near Havana, an increase in overbank storage of about 20 percent yields the results shown in Figure 5-13; an increase in storage of about 40 percent yields those shown in Figure 5-14. Both changes are only minor increases in storage area because the overbanks are confined by levees.

(3) By varying Manning's *n* with flow the reproduction of stage is improved; see Figure 5-15. The model still does not reproduce the initial time steps, but the disagreement is probably caused by the initial conditions.

(4) The final calibration consists of fine tuning the flow-roughness relation and the adjustments in storage. The event selected should be an extension of the event chosen for the initial calibration. For the Illinois River example, the final calibration was performed for the period from 15 Nov 1982 to 15 Sep 1983. The event includes high flow and low flow and a second major flood in May 1983. Figure 5-16 shows the reproduction of stage at Havana during the period. The model parameters required only slight adjustment to better simulate low flow.

*c. Verification.* The calibrated model should be verified against two or more periods which include significant events. The periods should be long, approaching one year, so that seasonal effects can be detected. Figure 5-17 shows the reproduction of the 1974 observed data on the Illinois River.

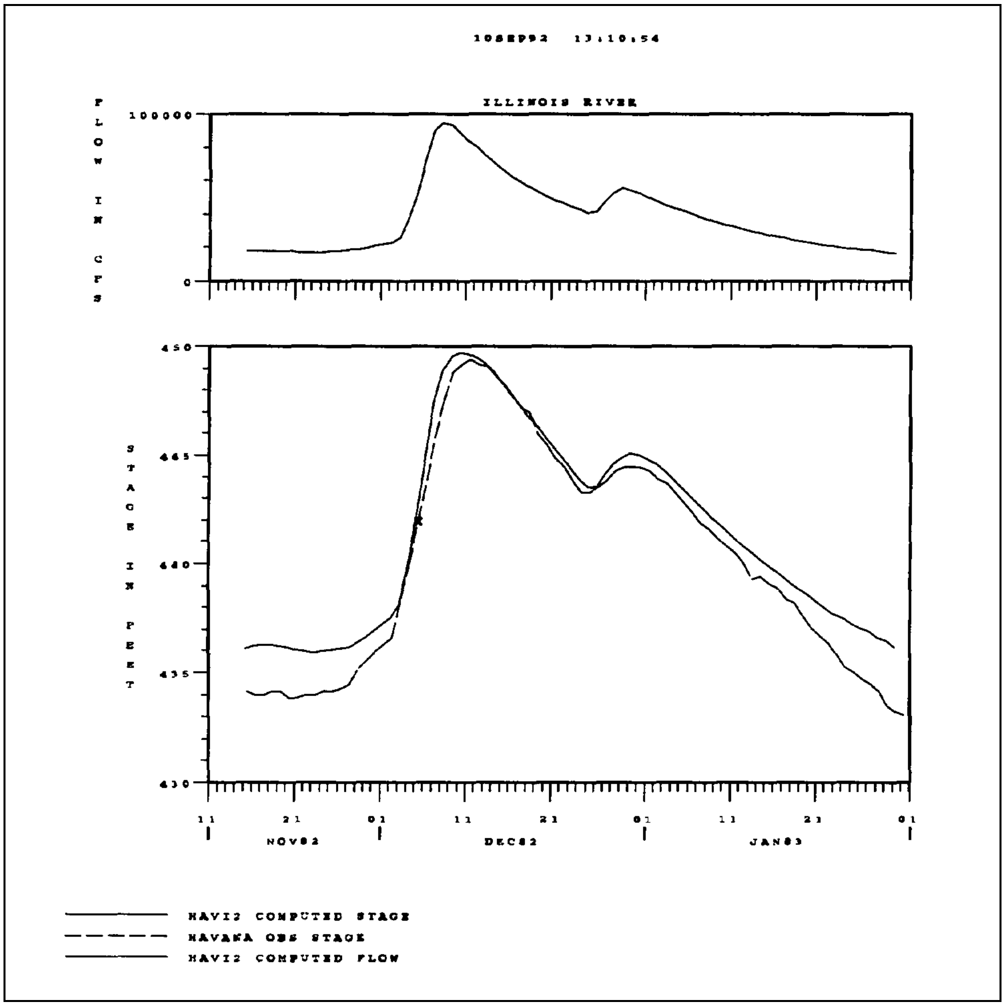


**Figure 5-12. Hydrographs for the Illinois River at Havana after initial calibration**

## 5-10. Example Applications of Unsteady Flow Models

Numerous applications, in addition to those presented above to illustrate the use of unsteady flow models, have been performed; the following is a brief summary. The one-dimensional unsteady flow program DWOPER, developed by the National Weather Service, has been used to simulate flood wave movement through the Central Basin of the Passaic River in New Jersey. This was a complex routing problem because of flat gradients and flow reversals that were involved (U.S. Army Corps of Engineers 1983). The one-dimensional unsteady flow model UNET has been applied to a 90-mile long reach of the Red River of the North to improve analysis of flooding on this river. The study reach was characterized by agricultural levees and other flow controlling features on a wide, flat floodplain (U.S. Army Corps of Engineers 1990c). Cunge et al. (1980) present several examples of applications to complex natural river systems. A study of potential mudflow movement in Castle Creek, near Mount Saint Helens was performed (U.S. Army Corps of Engineers 1990e) using the NWS DAMBRK model (Fread 1988).

*Section II*
*Theory of Routing Models*

## 5-11. Introduction

*a. General.* This section describes, in a one-dimensional context, the physical characteristics of flood waves passing through a reach of channel. An overview of

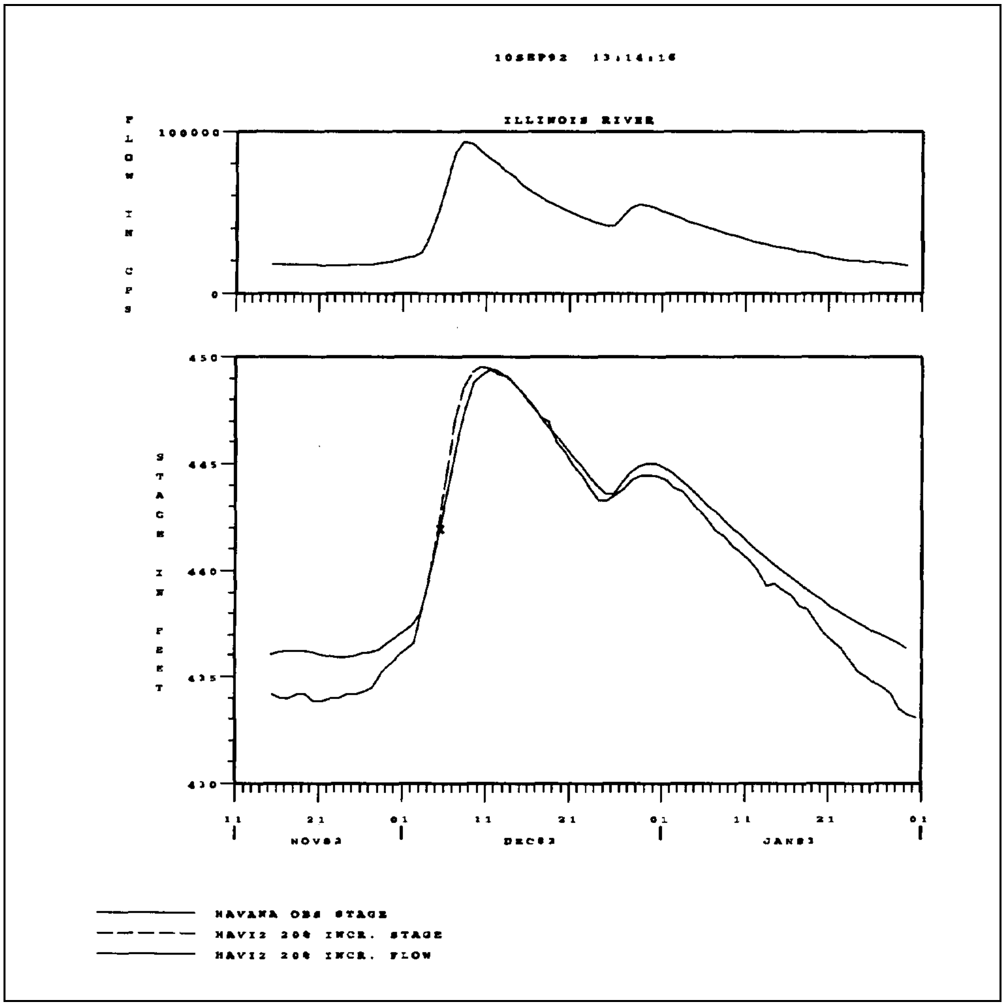


**Figure 5-13. Hydrographs for the Illinois River at Havana with overbank storage increased by 20 percent**

prediction techniques is presented: first hydraulic techniques, which simulate the wave motion by solving the mathematical equations governing the unsteady flow in the reach, and then hydrologic techniques, which compute outflow hydrographs directly from predetermined reach characteristics and a given inflow hydrograph. The effects that the assumptions characterizing a model have on its applicability are discussed.

*b. Hydrologic routing versus hydraulic routing.* In the nineteenth and early twentieth centuries, the approaches used to analyze problems associated with the movement of water were fragmented among different professions in accord with the area of endeavor affected by the particular case of water motion. The assumptions developed to allow solution of these complex problems varied widely in the different fields in accord with the inventiveness of the researcher and were generally unrelated. Classical hydrodynamicists studied the mathematics of potential flow of a perfect fluid, which water under certain circumstances imperfectly imitates. Mathematicians studied laminar flow, a turbulence-free phenomenon in which fluid mixing takes place only on a molecular level. Laminar flow is rarely seen in rivers; the high Reynolds numbers and boundary roughness of a typical river make turbulent flow the norm. Hydraulic engineers developed empirical formulas for head loss in turbulent flow in pipes. Because of the greater complexities of open channel flow, engineers devised assumptions and computational schemes to be as simple as possible for analyzing river flows.

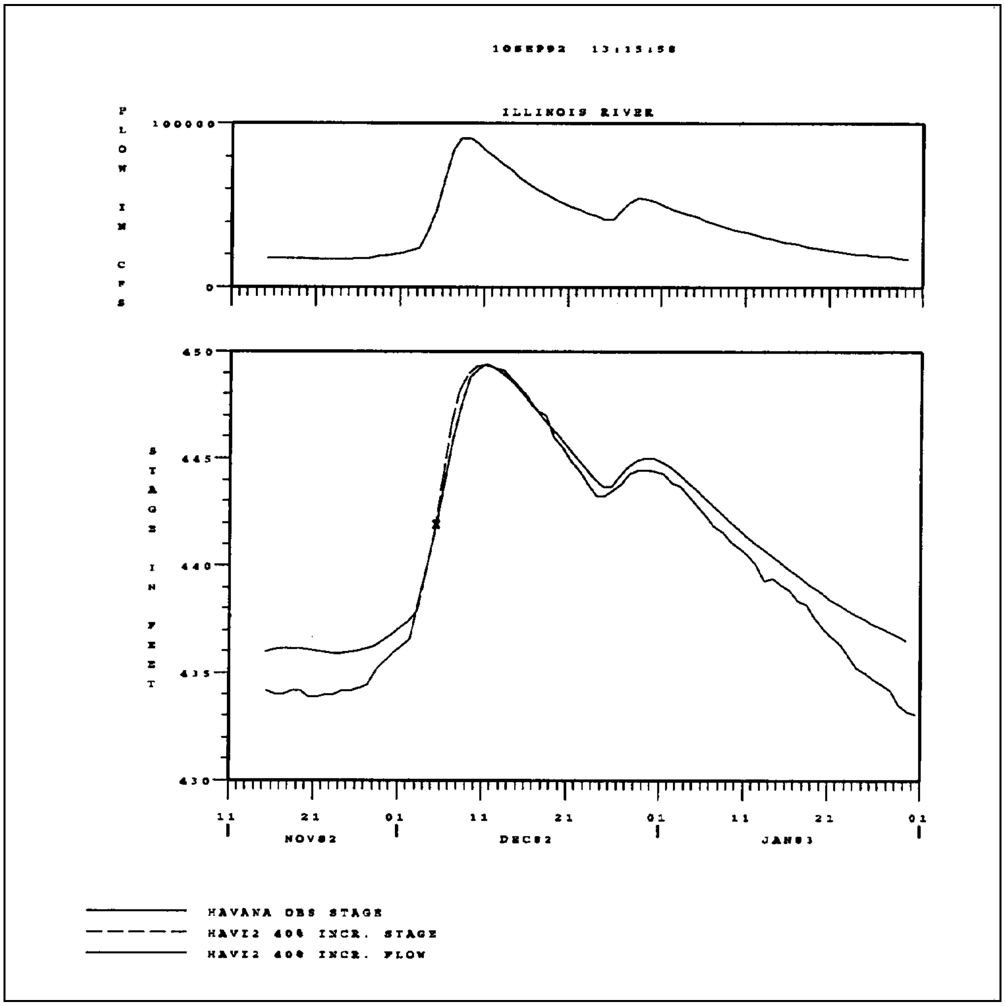


**Figure 5-14. Hydrographs for the Illinois River at Havana with overbank storage increased by 40 percent**

(1) This section seeks to relate the so-called hydrologic and hydraulic approaches to flood routing. The hydrologic approaches, which are simpler to use but harder to defend theoretically, are viewed from the point of view of the hydraulic approaches, which are better grounded in basic theory but relatively difficult to apply.

(2) The aim of both approaches is the same: to determine the response in a given reach of a watercourse to a given inflow sequence (usually a flood hydrograph), and, both recognize the physical principle of conservation of mass. They both seek to account, at all times, for all of the volume of water initially in the stream and that of the inflow(s) and outflow(s). The volume of water stored in a reach varies with time as a flood wave passes through.

(3) Mathematically, with *I(t)* representing an inflow sequence (hydrograph), *T(t)* the net lateral inflow along the length of the reach (tributary inflow minus infiltration, etc.), *O(t)* the outflow hydrograph, and *S(t)* the volume of water (storage) between the inflow and outflow sections, the principle of conservation of mass can be written:

$$I(t) + T(t) - O(t) = \frac{dS(t)}{dt} \tag{5-1}$$

(4) The argument, *t*, is explicitly stated to underscore the premise that the equation holds true at each instant of time. With the inflow hydrograph given, and with the tributary hydrograph given, estimated, or

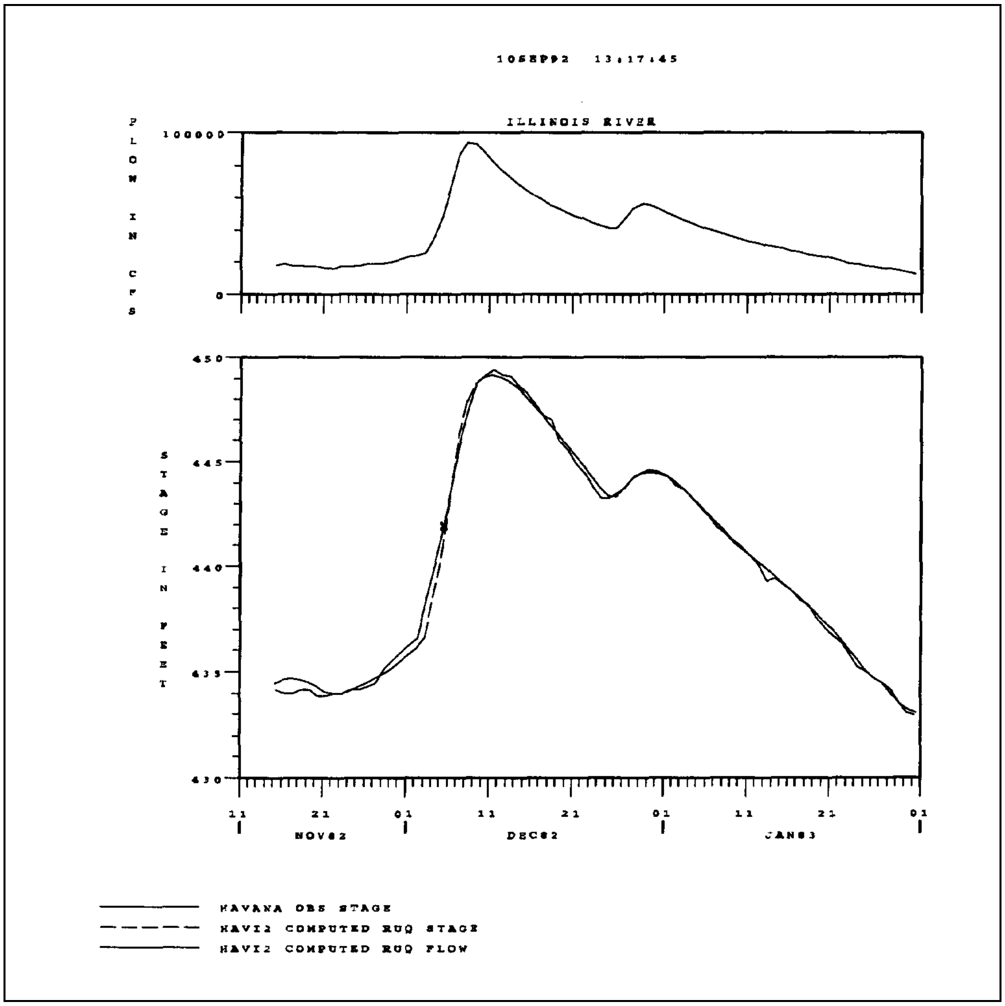


**Figure 5-15. Hydrographs for the Illinois River at Havana with adjusted flow-Manning's *n* relationship**

neglected, the outflow hydrograph can be computed if the relation of the storage to the hydrographs is also known. It is on this issue, the relationship between the geometrical quantity, storage, and the kinematic quantities, discharge hydrographs, that the hydrologic and hydraulic approaches differ.

(5) The hydrologic techniques focus attention on discharge hydrographs. The outflow discharge hydrograph constituting the response of the reach to the inflow hydrograph is computed directly, and after that is done, the water levels in the reach are somehow related to the discharges. To achieve such a direct solution for the outflow hydrograph, a storage versus flow relation is assumed, either empirically on the basis of flood events of record for the reach, or theoretically on the basis of some simplifying physical assumption. In the most empirical of the hydrologic techniques, the storage is not even considered; inflow hydrographs are manipulated by an averaging technique flexible enough to allow matching of computed and measured outflow hydrographs.

(a) Furthermore, in hydrologic methods, the study reach is treated as a whole. Even if the reach is broken into subreaches, as some of the techniques propose, it is assumed that the outflow hydrographs can be determined sequentially, from upstream to downstream. The outflow hydrograph of one subreach serves as the inflow hydrograph for the neighboring downstream subreach.

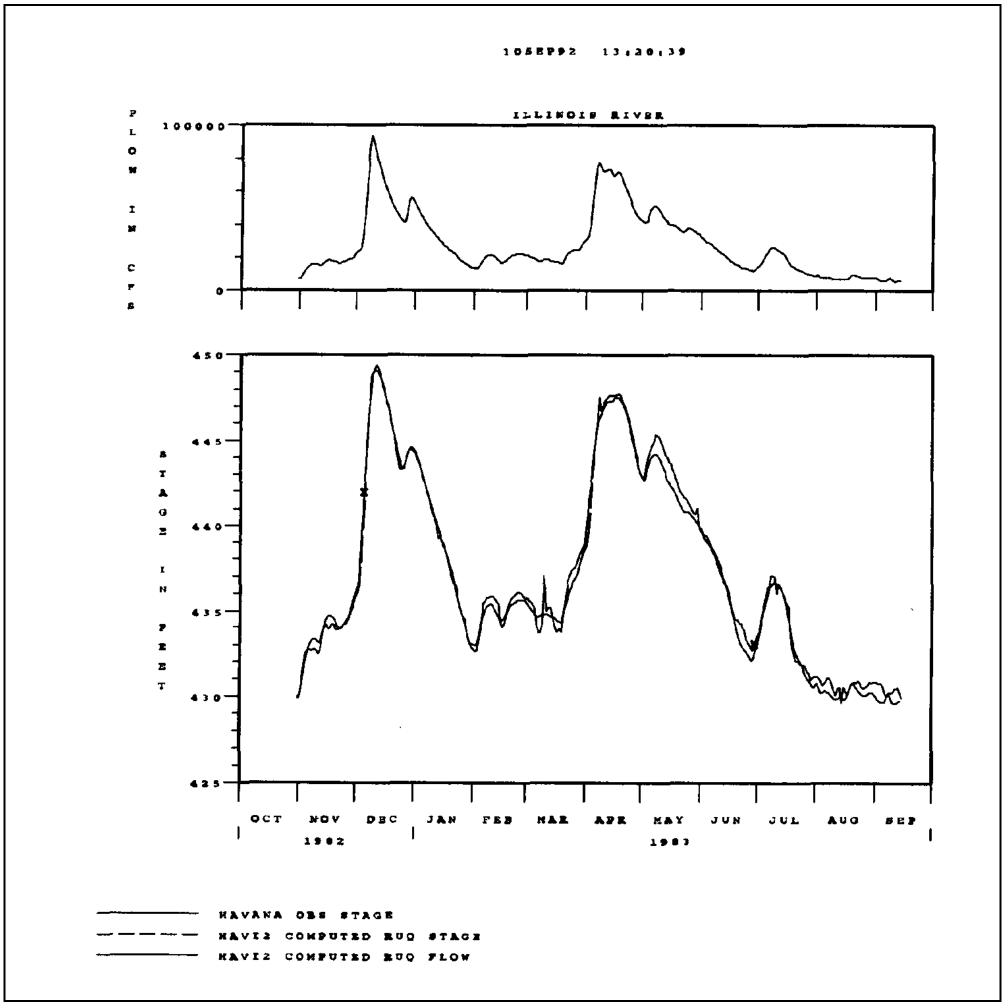


**Figure 5-16. Hydrographs for the Illinois River at Havana with flow-Manning's *n* relationship adjusted to reproduce the 1983 flood**

(b) In this approach, time is the only variable, and the mathematics of the simulation reduce generally to an ordinary differential equation. This equation relates the sought after time variation of the outflow to the given time variation of the inflow and to the given response characteristics of the reach, e.g. a storage versus flow relationship. The hydrologic techniques typically solve this differential equation numerically, i.e. algebraically, through the use of finite-sized time steps.

(6) The hydraulic approaches explicitly recognize, in addition to the physical principle of mass conservation, a second physical principle, one or another form of conservation of momentum. If, then, an assumption is made regarding the shape that graphs of the variation of stage and discharge along the reach would have, absolute values for both profiles can be found. The usual assumption is that the shape of the stage and discharge profiles cannot be given *a priori* for the reach as a whole. It must be broken into a sufficient number of distance steps so that the shape of depth and discharge variation in each can be assumed to be a straight line. For this reason, the hydraulic techniques generally require a determination of depth and discharge at a sequence of stations within the reach, even if the conditions are in fact sought at only one point.

(a) As a result, a characteristic feature of hydraulic approaches is the calculation of flow variables in the interior of the study reach, even if they are not of special interest. For example, to arrive at the outflow

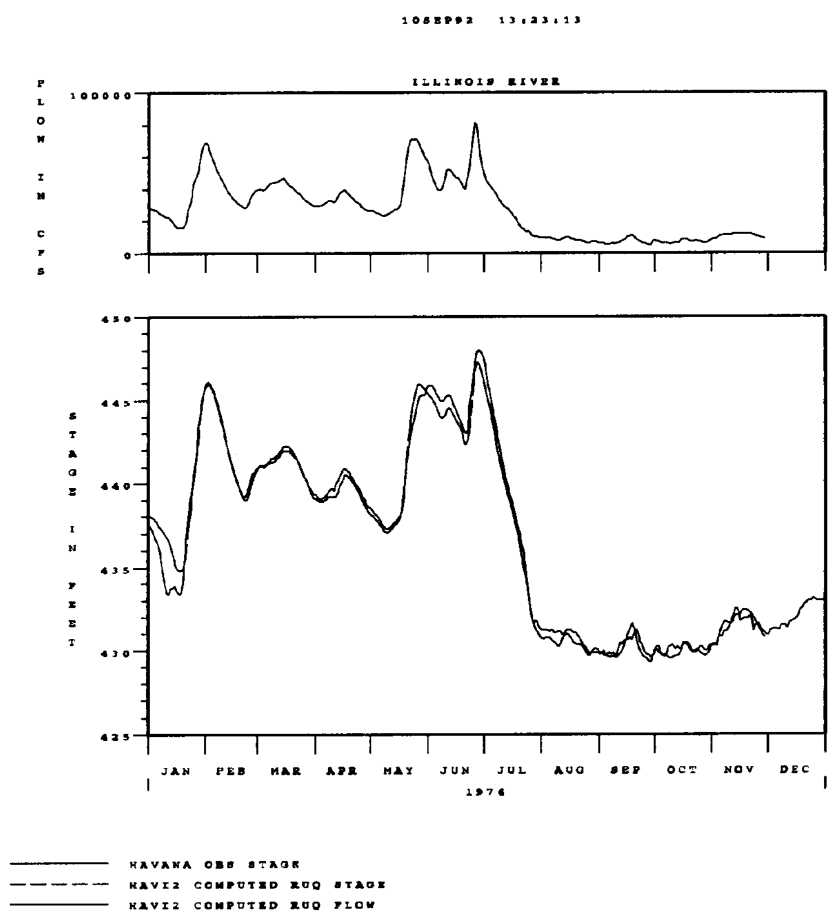


**Figure 5-17. Verification of the Illinois River model against 1974 observed data**

hydrograph for a reach subject to a given inflow hydrograph at its upstream end, the hydraulic methods compute water surface elevations and discharges at a sequence of stations in the interior of the reach. The desired hydrograph is computed along with all interior hydrographs, and stages in the reach are routinely determined as part of the solution. In another example, the calculated advance of a dam-break flood wave is a by-product of calculations of flow conditions in the interior of the wave.

(b) In the limit, as the number of distance steps increases indefinitely, while the size of each is correspondingly reduced, the governing physical principles lead to partial differential equations in distance along the channel and time. The dependent variables are the time dependent profiles of depth and discharge (or depth and discharge hydrographs at all stations in the reach). These partial differential equations are generally solved numerically, algebraically, in finite-sized distance, and time steps with the aid of high-speed electronic computers.

(7) The hydrologic techniques are often easier to apply than the hydraulic techniques and are usually associated with quicker, less troublesome, computations. Hydraulic methods require a description of the geometry and roughness of the reach usually defined by cross sections and reach lengths. Those hydrologic methods which use past flood hydrograph records to estimate the response of the reach bypass such detailed analysis of the physical characteristics of the reach; the lumped effect of its physical characteristics is assumed to be incorporated

into the measured responses. And if, in fact, the reach does behave sufficiently like the calibration events for the flood being studied, the hydrologic approach may be nearly as accurate as any of the hydraulic routing schemes for determining discharge. The difficulty, of course, is in establishing the storage versus flow relation pertinent to the subject flood.

## 5-12. Unsteady Flow Model

*a. Unsteady flow equations.* Derivations of the unsteady flow equations are presented in numerous references. Chow (1959), Fread (1978), and User's Manual for UNET (U.S. Army Corps of Engineers 1991b)] are three of such references. They can be obtained from the two-dimensional equations presented in Chapter 4 by assuming that the dependent variables only change in one direction, *x*, and that direction is along the river axis rather than being a cartesian coordinate. Common formulations of the equations are as follows:

Equation of continuity

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} + \frac{\partial S}{\partial t} = q_L \qquad (5\text{-}2)$$

Equation of momentum

$$\frac{\partial Q}{\partial t} + \frac{\partial (QV)}{\partial x} + g\,A\left(\frac{\partial z}{\partial x} + S_f\right) = q_L\,V_L \qquad (5\text{-}3)$$

where

$Q$ = flow
$A$ = active flow area
$S$ = storage area
$q_L$ = lateral inflow per unit flow distance
$V$ = $Q / A$ = average flow velocity
$g$ = acceleration of gravity
$z$ = water surface elevation
$S_f$ = friction slope
$V_L$ = average velocity of the lateral inflow
$x$ = flow distance
$t$ = time

(1) The assumptions implicit to the unsteady flow equations are essentially the same as those for the steady flow equations: (a) the flow is gradually varied; that is, there are no abrupt changes in flow magnitude or direction; (b) the pressure distribution is hydrostatic; therefore, the vertical component of velocity can be neglected. This means, for example, that the unsteady flow equations should not be used to analyze flow over a spillway, and (c) the momentum correction factor is assumed to be 1.

(2) The magnitude of each of the terms in the momentum equation plays a significant role in the hydraulics of the system. The terms in equation 5-3 are:

$$\frac{\partial Q}{\partial t} = local\ acceleration$$

$$\frac{\partial (QV)}{\partial x} = advective\ acceleration$$

$$\frac{\partial z}{\partial x} = water\ surface\ slope$$

$$S_f = friction\ slope$$

The water surface slope can be expressed as

$$\frac{\partial z}{\partial x} = \frac{\partial h}{\partial x} - S_o \qquad (5\text{-}4)$$

in which *h* is the depth and

$$\frac{\partial h}{\partial x} = pressure\ term$$

$$S_o = bed\ slope$$

The roles of these terms are discussed below.

*b. Weaknesses of the unsteady flow equations.*

(1) Friction slope is the portion of the energy slope which overcomes the shear force exerted by the bed and banks, and it cannot be measured. To quantify the friction slope, the Manning or Chezy formulas for steady flow are used:

Manning's Equation

$$S_f = \frac{Q|Q|n^2}{2.21A^2R^{4/3}} \qquad (5\text{-}5)$$

where

$n$ = Manning's $n$ value
$R$ = hydraulic radius

Chezy's Equation

$$S_f = \frac{Q|Q|}{A^2C^2R} \tag{5-6}$$

in which $C$ is the Chezy coefficient. Note the use of the absolute value of discharge; this keeps the sign of $S_f$ proper for flow reversals.

(2) Equations 5-5 and 5-6 are semi-empirical equations for steady flow, but they also produce acceptable results for unsteady flow. Other equations have been proposed for estimating the friction slope Einstein (1950), Simons and Sentürk (1976), and ASCE (1975). Typically, these equations are logarithmic and contain sediment parameters. Most modelers have avoided these equations because they are computationally inconvenient, requiring an iterative solution to solve for the friction slope within each time step.

*c. Force exerted by structures.* Bridge piers, embankments, dams, and other hydraulic structures exert a force on the flow which is not considered in the momentum equation presented above. To illustrate this force, consider submerged flow over a broad crested weir as shown in Figure 5-18. The unequal pressure distribution on the upstream and downstream faces exerts a net force in the upstream direction on the flow. This force is not included in the friction term, nor is it included by the pressure force from the bank which is included in the water surface slope term. If the force is not included in the momentum equation, the computed swell head upstream of the structure will be too small. Moreover, the force is seldom quantified. The emphasis of research has been to quantify the energy loss through structures, which is useful for computing the swell head for steady flow.

(1) Modelers Fread (1978), and Barkau (1985) have proposed augmenting the momentum equation with an additional slope term based on the energy loss:

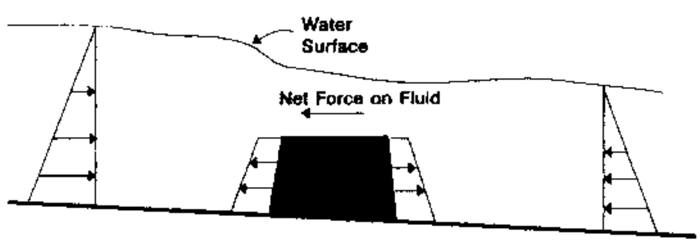


**Figure 5-18. Exterior forces acting on a control volume of fluid flowing over a broad crested weir**

$$S_h = \frac{h_L}{dx} \tag{5-7}$$

in which $h_L$ is the head loss due to the force and $dx$ is the distance over which the loss occurs.

(2) Since energy loss is obtained by integrating force applied over distance, Equation 5-7 estimates an additional energy slope to overcome the force. The added slope produces the correct swell head upstream of the structure. The augmented momentum equation now becomes:

$$\frac{\partial Q}{\partial t} + \frac{\partial (QV)}{\partial x} + g\,A\left(\frac{\partial z}{\partial x} + S_f + S_h\right) = q_L\,V_L \tag{5-8}$$

*d. Subcritical and supercritical flow.* The unsteady flow equations are wave equations. Disturbances propagate according to the rate

$$\frac{dx}{dt} = V \pm c \tag{5-9}$$

where

$c$ = the celerity of a gravity wave
$c$ = $(gD)^{1/2}$
$D$ = hydraulic depth

(1) If $V < c$, the flow is subcritical, and disturbances move both upstream and downstream. Hence, a disturbance downstream, such as a rise in stage, propagates upstream. If $V > c$, the flow is supercritical, and the velocity sweeps all disturbances downstream. Hence, a stage disturbance downstream is not felt upstream.

(2) Equation 5-9 has profound implications for the application of the unsteady flow equations. Subcritical flow disturbances travel both upstream and downstream; therefore, boundary conditions must be specified at both the upstream and downstream ends of the routing reach. For supercritical flow, the boundary conditions are only specified at the upstream end.

(3) Near critical depth, the location for the boundary conditions is changing; hence, the flow and the numerical solution may become unstable. Instability when the depth is near critical is one of the greatest problems encountered when modeling unsteady flow. Most streams which are modeled with unsteady flow are subcritical at higher stages but, at lower stages the pool and riffle sequence usually dominates flow. Supercritical flow can occur at the riffles. Because unsteady flow models simulate the full range of flow, the models can become unstable during low flows.

*e. Numerical models.* An unsteady flow model (also called a dynamic wave model) solves the full momentum and continuity equations. Forces from all three sources (gravity, pressure, and friction) and the resulting changes in momentum (local and advective accelerations) are all explicitly considered along with mass conservation. If the assumption of one-dimensional flow is justified, and the discretization of flow variables introduces little error, then the simulation results are as accurate as the input data. Unsteady flow models differ in their underlying physical assumptions, in the way in which the real continuous variation of flow variables with space and time is approximated or represented by discrete sets of numbers, and in the mathematical techniques used to solve the resulting equations. Other differences reflect the range of different steam networks, channel geometries, control structures, or flow situations that the model is designed to simulate. For example, not all dynamic wave models are equipped to handle supercritical flow; a typical indication of failure is oscillating water surface profiles and an aborted execution. There are also differences (which can strongly effect study effort) in input data structure, user operation, documentation, user support, and presentation of results.

(1) Such a model can accurately simulate flows in which acceleration plays an important role, such as flood waves stemming from sharply rising hydrographs such as a dam break flood; disturbances of essentially still water, for example the drawdown of water in the reservoir behind a ruptured dam; and seiching, which is a long period longitudinal oscillation of water. Another example of a situation that can be modeled only by a dynamic wave model is the reflection of a dam break flood wave from a channel constriction.

(2) As the bed slope becomes small, it becomes less important than the water surface slope and the acceleration terms play a greater role. The looped rating curve is an example of this phenomenon. For streams on a low slope, the rising limb of the hydrograph passes at a lower stage than the falling limb for a particular discharge. The loop for the Illinois River at Kingston Mines during the December 1982 flood is shown in Figure 5-19. The flow and stage hydrographs were shown in Figure 5-8.

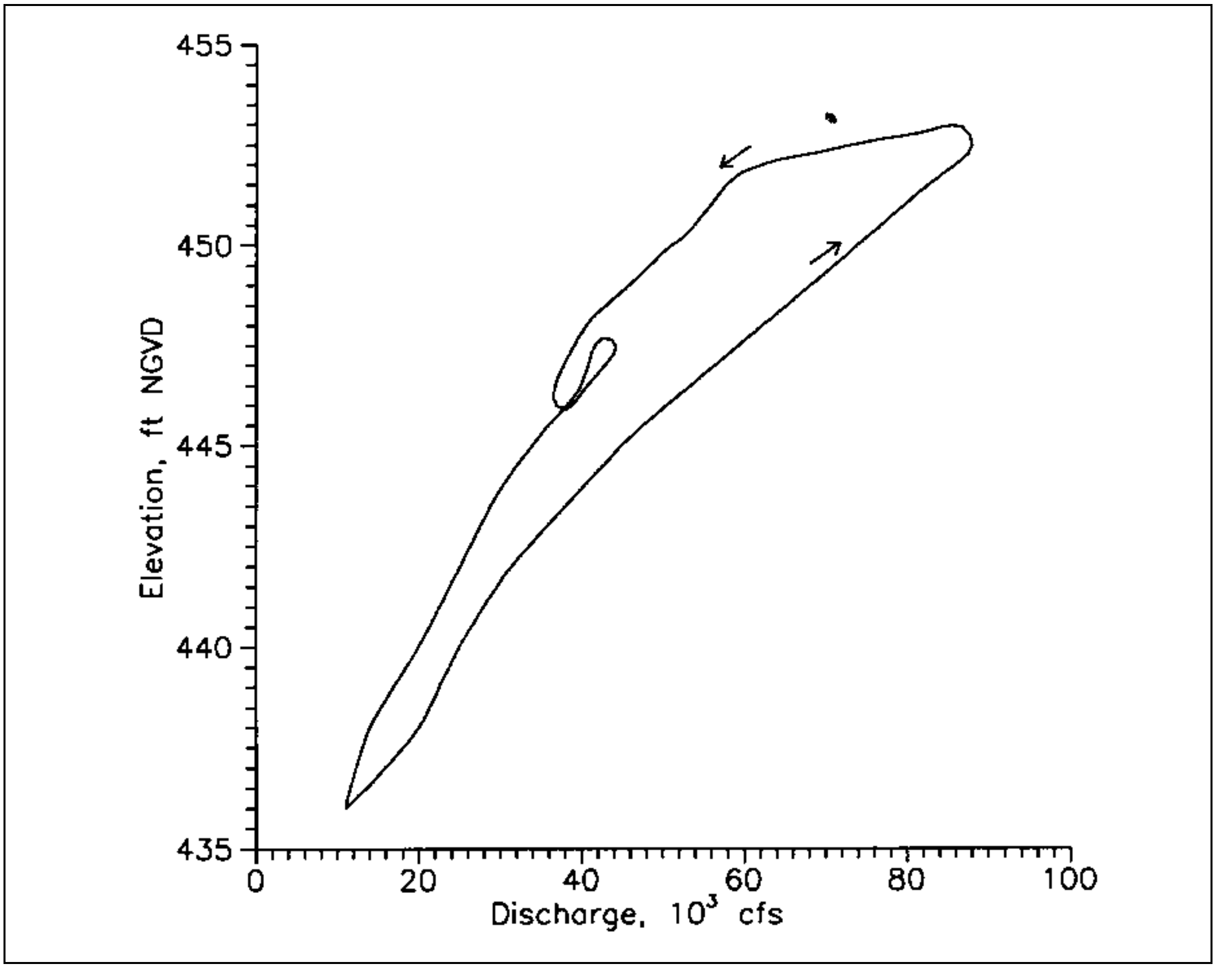


**Figure 5-19. Looped rating curve for the Illinois River at Kingston Mines, 15 Nov 82 to 31 Jan 83**

The peak flow always precedes peak stage. The loop can be explained with the help of Figure 5-20. The slope of the water surface is greater on the rising limb than on the falling limb, thus the flow is accelerating on the rise and decelerating on the fall.

(3) If the flow changes rapidly, then the acceleration terms become important regardless of the slope of the bed. The advective acceleration term diffuses the discharge downstream; it lengthens and attenuates any rapid change in discharge. Figure 5-21 shows a test of routing a rapidly rising and falling hypothetical hydrograph through a channel of unit width using an unsteady flow model. In 8,000 feet the peak discharge had attenuated by over a third and the hydrograph had lengthened dramatically. This is typical of dam break type waves.

*f. Numerical approximations.* Discretization, the representation of a continuous field of flow by arrays of discrete values, is a major concern in the construction of unsteady flow models. The choice of scheme influences the ease of writing, correcting, and modifying the program; the speed at which the program executes; accuracy of the solution, including satisfaction of volume conservation, momentum conservation, and computation of proper wave velocities; the robustness of the model; and ultimately, its stability.

(1) Explicit solution schemes allow the computation of flow variables at the end of a time step at one point in the channel, independent of the solution for neighboring points. Implicit schemes solve for the flow variables at the end of a time step at all points in the channel simultaneously. The former are easier to program and maintain, but require small time steps to avoid computational instability. The required size of the time steps for usually much less than that needed to resolve the rates at

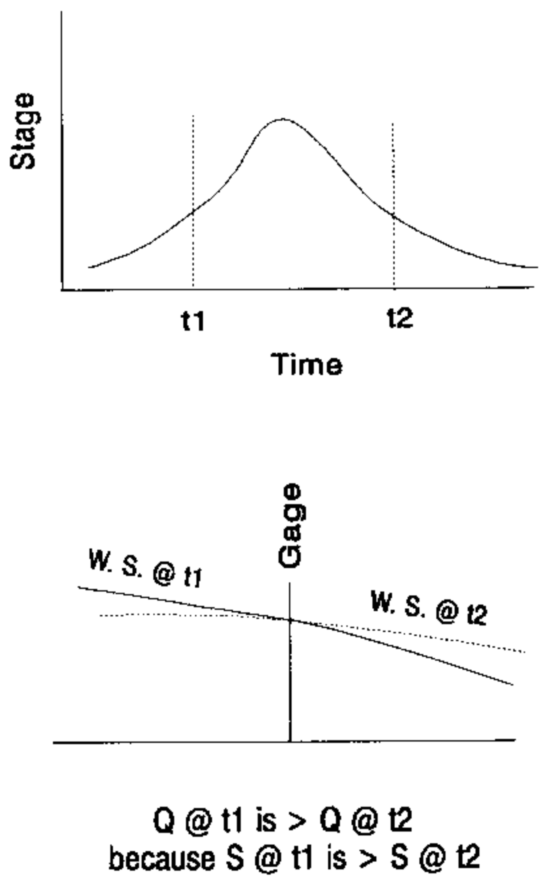


Figure 5-20. Explanation for looped rating curve effect

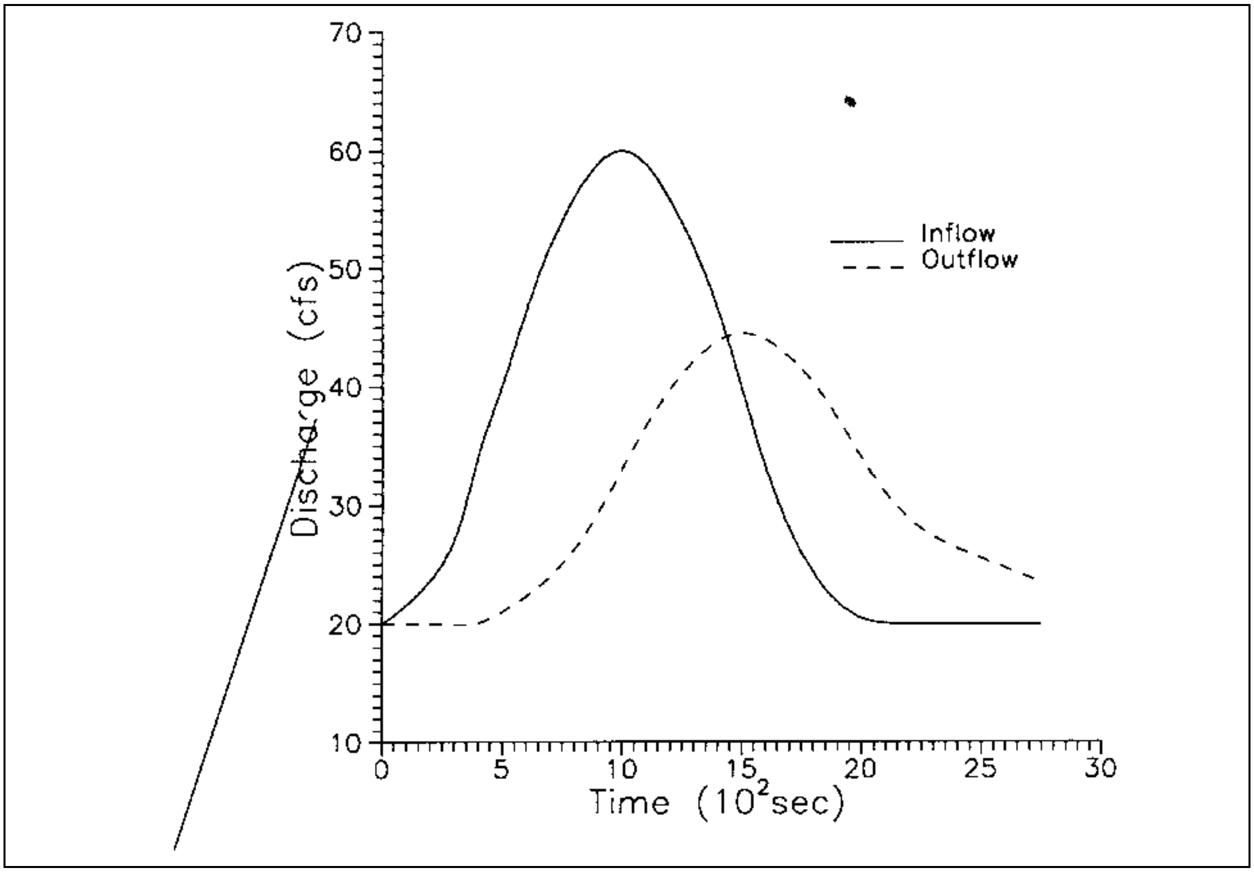


**Figure 5-21. Attenuation from a hypothetical dam break type flood routed 8,000 feet downstream through a channel of unit width**

explicit schemes is which changes are occurring to the flows at reach boundaries. This can lead to a very inefficient solution. The time steps for implicit schemes are, theoretically, dependent only on accuracy criteria and can be many times larger than in explicit schemes. Implicit models appear, further, to be generally more robust.

(2) Most of the successful models available today use an implicit finite difference scheme (Fread 1978, 1988; Shaffranek et al. 1981; Johnson 1982; U.S. Army Corps of Engineers 1991b).

## 5-13. Diffusion Model

For some flow conditions the water surface slope and the friction slope are nearly equal and the momentum equation becomes

$$\frac{\partial z}{\partial x} \approx -S_f \qquad (5\text{-}10)$$

This is the diffusion wave, or zero-inertia approximation. Forces from all three sources are assumed to be in equilibrium, so that the acceleration is zero. If the sum of local acceleration (a measure of unsteadiness) $\partial Q/\partial t$ and advective acceleration (a measure of nonuniformity) $\partial(QV)/\partial x$ is small compared to the sum of weight (i.e., gravitational) and pressure components, this model is capable of producing a simulation virtually as realistic as the dynamic wave model. This is often the case for flows at a low Froude number.

*a. Assumptions.* Local and advective accelerations are often of similar magnitude and opposite sign; their sum is typically smaller than either one alone.

*b. Nonuniformity.* Only when the nonuniformity of the flow is primarily the result of nonuniform channel geometry, rather than because of unsteadiness, can the local acceleration be small compared to advective acceleration.

(1) The neglect of all acceleration terms in the diffusion model has advantages and disadvantages. A major advantage is a more robust model, because the acceleration terms are sometimes the source of computational fragility, especially in a near-critical or supercritical flow. To a diffusion model, all flows are infinitely subcritical.

(2) The disadvantages include the inability to simulate certain kinds of flow, seiching is infinitely damped, and bores are imperfectly imitated by relatively gradual rises in water surface elevation.

(3) The magnitude of the error in outflow hydrograph prediction for typical inflow hydrographs depends on the channel and inflow hydrograph characteristics.

## 5-14. Kinematic Wave Model

*a. Slope.* If the slope of the bed is relatively steep and the change in discharge is moderate, the pressure term and the acceleration terms become small compared to the bed and friction slope terms. Hence, the friction slope and the bed slope are approximately in balance as shown in Equation 5-11.

$$S_f \approx S_o \tag{5-11}$$

This is called the kinematic wave approximation, and the flow can only be routed downstream. The water surface elevation at each section can be calculated from Manning's equation or obtained from a single-valued rating curve for any discharge. There are no backwater effects. The physical assumptions in this approximate method are often justified in overland flow or steep channels if the flow is well established so that there is little acceleration.

*b. Limitations.*

(1) The method is patently useless in horizontal channels, because there is drag but no streamwise weight component. It typically overestimates water depth in channels of small slope. As a rule of thumb, the kinematic wave approximation may be applicable for slopes greater than 10 feet per mile, depending upon the shape of the hydrograph. Experience has shown that kinematic wave should not be used when analyzing flows in rivers.

(2) A characteristic feature of flood wave behavior computed with this method is that, in the absence of lateral inflow/outflow, there is no subsidence of the crest. Certain numerical schemes introduce a spurious numerical subsidence, but that cannot be used rationally to model real subsidence. The phenomenon of kinematic shock allows flood wave subsidence within the context of kinematic wave theory, but does not model real subsidence. When subsidence is important, a kinematic wave model should not be used.

(3) The major advantage of kinematic wave is that it displays no computational problems at critical depth.

## 5-15. Accuracy of Approximate Hydraulic Models

Numerical criteria are presented in Ponce (1989) for estimating the relative accuracy of approximate models. Some of the criteria are based on the relative magnitude of neglected terms in the unsteady flow equations (5-3 and 5-4). Others, dealing with hydrologic methods, are concerned with subreach length relative to length of the flood wave. Still others stem from the results of comparative tests.

*a. Kinematic versus diffusion.* According to Ponce (1989), kinematic and diffusion wave models may be used in reaches with little or no downstream control. The diffusion wave has a wider range of applicability than the kinematic wave and should be used unless a strong case can be made for the latter. Ponce suggests the following criteria for determining applicability of the two methods:

The kinematic wave model can be used if

$$\frac{T_r S_o u_o}{d_o} > 85 \tag{5-12}$$

The diffusion wave model can be used if

$$T_r S_o \sqrt{\frac{g}{d_o}} > 15 \tag{5-13}$$

where

$T_r$ = hydrograph time of rise
$S_o$ = equilibrium energy slope (or bottom slope for channel of regular cross section)
$u_o$ = average velocity
$d_o$ = average flow depth
$g$ = acceleration of gravity

*b. Data requirements.* These depend on the nature of the method and are described in the sections which follow and in Appendix D. In general, hydraulic models require channel geometry, boundary roughness, the initial state of the water in the channel, and an upstream flow hydrograph.

(1) An upstream boundary condition with its time variation, such as a discharge or depth hydrograph, must be specified, as must be the tributary inflows or outflows. In the special case of supercritical flow at the upstream end of the reach, both depth and discharge must be given to a dynamic wave model.

(2) With the dynamic wave and diffusion models, either a depth or discharge hydrograph is required at the downstream end. In the special case of supercritical flow at the outlet (dynamic wave model), no downstream boundary condition can be given.

(3) No downstream condition can be given to the kinematic wave model, nor to any of the hydrologic models, as they all employ "marching" solutions, progressing from upstream to downstream.

## 5-16. Muskingum-Cunge Model

While the origin of this model is the Muskingum method, a hydrologic technique, its theoretical basis and application, typically to a large number of subreaches, suggest that its classification be as a hydraulic method. As such, it is a subset of the diffusion approach; the additional assumption, linearization about normal depth at the local discharge, leads to problems with accuracy at low values of bottom slope and precludes analysis of flows in which backwater effects play a role. Its advantages over the diffusion approach are not known at this time; comparisons might prove it to be a more robust model.

## 5-17. Hydrologic Routing Schemes

Hydrologic routing focuses on the study reach as a whole; there is still need for two equations to solve for the two related variables, water surface elevation and discharge, even if these are required at just one location. The principle of mass conservation supplies one of the required equations but, instead of applying the momentum equation in the interior of the flow, a different theoretical or empirical relation provides the second equation. A summary discussion is presented below.

*a. Average-lag methods.* Two significant features of flood hydrographs have long been observed in many rivers. Reflecting the wave-like character of flood behavior, hydrographs at successive stations are displaced in time; peaks, for example, occur later at each successive downstream station. In other words, downstream hydrographs lag upstream hydrographs. The second observation is that, usually, hydrograph peaks exhibit subsidence; that is, a decrease in peak value with distance downstream if there is no significant tributary inflow.

(1) Such behavior is observed in the results of the so-called average-lag methods, empirical techniques based on averages of inflow hydrograph values lagged in time. Averages of groups of hydrograph values are always less than the largest of the group unless all members of the group are equal; in particular, the average of values in the vicinity of the peak will be less than the peak itself. Freedom in choosing the time spacing of points on the inflow hydrograph, the number of points to include in the average, the weighting coefficients defining the average, the number and length of subreaches to which to successively apply the technique, and the travel time for the hydrograph in each subreach; i.e., the amount of time to lag the hydrograph, often provides enough flexibility to allow a match of lagged average reach-outflow hydrographs with observed ones in a calibration event. Many years of familiarity with a reach of river and with the observed hydrographs can facilitate choosing the parameters of such a method for a reasonably good fit of computed and measured hydrographs, but satisfactory routing under different circumstances would have to be considered fortuitous. There are many ways in which hydrograph values can be averaged and lagged. There is no theoretical reason to favor one over another.

*b. Progressive average-lag method.* This technique as found in EM 1110-2-1408 also known as Straddle-Stagger (U.S. Army Corps of Engineers 1990a), is the most empirical of these methods. It provides hydrographs which exhibit subsidence and time lag, and these can often be made to match observed hydrographs through adjustment of the arithmetic parameters of the method.

(1) The reach is treated as a whole; subreach length equals reach length. Equal weight is given to the inflowing hydrograph values in determining their average. The time period over which averaging occurs is centered on the inflow value being routed; i.e., the one at a lag-time duration earlier than the time pertinent to the outflow hydrograph value. The constant time interval used to define the inflow hydrograph, the number of points used for averaging, and the lag time (outflow value time minus routed inflow value time, expressed as an integer number of time intervals) are chosen by trial and error for a best fit with observations.

(2) The hope in using this method is that the storage/hydrograph relation that exists for the reach in the calibration event is reflected in the arithmetic parameters determined, and that these will continue to be valid for the subject event. The lack of any theoretical basis for this hope makes the method unreasonable rather than approximate. The term approximate suggests that there is some control over the amount of error. But, in principle, the error in the computed subsidence for the subject event could be zero, plus or minus a hundred percent or more. Only if a series of calibration events lead to about the same parameter values in each case could one reasonably suppose that a subject event in the same reach with about the same inflow hydrograph as the calibration events, calculated with those values of parameters, would yield an outflow hydrograph of about the same accuracy as the calibration events. In general, the method is not recommended.

*c. Successive average-lag method.* In this technique (EM 1110-2-1408 1960), also known as the Tatum Method, each ordinate of the outflow hydrograph for a subreach is the numerical average of the routed inflow value and the preceding ordinate in the hydrograph. The ordinates of the inflow hydrograph are separated by constant time intervals, $\Delta t$, a parameter of the method. Subreach length is defined as the distance traveled by the flood wave in a time interval $\Delta t/2$, taken as the lag time. The outflow hydrograph for a subreach constitutes the inflow hydrograph for the next subreach, for which the procedure is repeated.

(1) Additional subreaches are introduced until the outflow for the subject reach has been determined. The number of subreaches constitutes another parameter of the method. The parameter values are chosen for a best fit with calibration hydrographs.

(2) A physical interpretation of the Tatum Method exists; it corresponds to a linear Modified Puls technique in which subreach storage is directly proportional to subreach outflow with the constant of proportionality $K = \Delta t/2$. Nonetheless, the method, like Progressive Average-Lag, must be considered empirical, and is not generally recommended.

*d. Modified Puls.* This approach is more rational than the average-lag methods, because it strives to solve the mass-conservation relationship (equation 5-2) by providing a second, storage versus flow, relation necessary to close the system.

(1) The method is characterized by a far-reaching physical assumption which, unfortunately, is often not warranted in rivers. The required storage versus flow relation stems from the assumption that there exists a unique relationship between storage in the reach and outflow from the reach. It is further assumed that this relationship can be found for the reach, either theoretically or empirically from past events; and that, once determined, applies to the study event. The mathematical form of the relationship is not important, a graph or table of numbers will suffice.

(2) An empirical relation can be found by measuring discharges as they vary with time during a calibration flood event at the inlet and outlet of the reach and applying the volume-conservation principle, (Equation 5-2). To the extent that tributary flow is accounted for, the relationship is valid for the event for which the information was recorded. To the extent that the relationship will continue to be valid for another event, or a different inflow hydrograph, it can be successfully used to predict outflow hydrographs for that event.

(3) A storage-outflow relation can be easily devised for a channel which is so large that the water surface remains level during the event to be simulated (a reservoir or "level pool") and if a discharge coefficient, theoretical or empirical, is available for the outlet. This is the physical circumstance for which the basic assumption of the Modified Puls method is valid.

(4) Hypothetical relationships between storage and outflow are sometimes derived for rivers from steady flow computations. Steady water surface profiles and, hence, water volumes, are computed in the reach for a sequence of discharges (outflows). The resulting table of

volumes as a function of discharge constitutes the storage/outflow relation. Such a relation ignores the effects of unsteadiness on the flood wave profile and hence on storage. The method can be successful if the local accelerations are negligible; i.e., if the reach is so geometrically nonuniform that advective accelerations from that source are large, and, at the same time, the rate of rise of the flood is so small that local and advective accelerations resulting from the unsteadiness are negligible in comparison.

(5) A potential source of major error with the Modified Puls method is that, in some flow circumstances, there is no physical relation between reach storage and outflow. The method does not account for the time changes in water flow that are transmitted as waves and not instantaneously from one end of the reach to the other. For example, a sharp increase in discharge at the upstream end of a reach produces a wave of increased depth that travels downstream at some velocity, generally somewhat greater than the water velocity. Thus, the storage in the channel starts to increase immediately, but the outflow is not affected at all until the wave finally arrives at the downstream end of the reach.

(6) The storage/outflow relation derived from a sequence of steady flows is unique; it plots as a single curve without hysteresis. But even a stage/outflow relation at a gaging station exhibits hysteresis in unsteady flow, with one branch of the hysteresis loop describing the function for the rising limb of the hydrograph and the other for the falling limb. This is due to the influence of local acceleration and its effect on water surface slope and advective acceleration. While a small amount of hysteresis is not of great concern, the hysteresis loop for a storage/outflow relation can be markedly more pronounced because of the traveling flood wave volume passing through the reach.

(7) In order to devise a more correct theoretical relation between storage and outflow than is possible using the entire reach as a unit (typically, the shape of the water surface within the reach is unknown), the reach may be broken into a number of subreaches. In each of these, the water surface is assumed level, or parallel to the bottom, and the outflow of a subreach is related to the depth through some uniform flow formula such as the Manning equation. As the number of subreaches is increased indefinitely, the scheme approaches that of the kinematic wave theory.

(8) Except for level-pool routing, the Modified Puls method should be used with caution, particularly for conditions outside the range of events used for calibration.

*e. Muskingum technique.* The assumption is made that the storage in a reach at some instant is related to both the inflow and outflow of the reach at that instant, which is more realistic than relating storage to outflow alone, as in the Modified Puls method. In the Muskingum technique storage is assumed to be in part directly proportional to inflow and in part directly proportional to outflow. The constants of proportionality can be determined either empirically from a study of known events or theoretically as in the Muskingum-Cunge technique. The major cause for concern in empirical derivations is that the subject simulation event may not produce the same wave profiles as the calibration event(s).

*f. Muskingum-Cunge technique.* In addition to the diffusion wave assumptions, the assumption is made that during the passage of the flood wave down the reach, departures from normal depth in the reach are not great. Then the proportionality constants in the Muskingum method can be determined theoretically. The diffusion equations are linearized about normal depth for some average condition in the reach and the results manipulated to yield the proportionality coefficients. The theoretical nature of the determination of the coefficients suggests that this is a hydraulic rather than hydrologic technique, especially, if the reach is broken up into a large number of subreaches to account for the unknown shape of the flood wave and to better schematize the boundary geometry. It is also discussed in section 5-16.

*g. Working R and D method.* This method is the same as the Muskingum method in that storage is assumed to be related to both inflow and outflow, but not necessarily proportional. Tabulated or graphed relations are envisioned. The method has more potential than Modified Puls (which can be considered a subset of the working R and D method) because it allows for the possibility that reach storage depends on inflow as well as outflow.

# Chapter 6
# Steady Flow - Water Surface Profiles

*Section I*
*Introduction*

## 6-1. Scope

This chapter is limited to a discussion of calculating rigid boundary, steady-flow, water-surface profiles. The assumptions, equations, and general range of application are presented in this section; data requirements, model development, special problems, and an example calculation follow in subsequent sections.

## 6-2. Assumptions of the Method

Computer programs used to compute steady, gradually varied flow water surface profiles are based on a number of simplifying assumptions. A thorough understanding of these assumptions is required before an adequate model of a study reach can be developed. Considerable engineering judgment is required in locating cross sections and preparing input data. The assumptions and how they affect program application follow:

*a. Steady flow.* Depth and velocity at a given location do not vary with time. This assumption requires that the flow remain constant for the length of time being considered. Of course, for natural rivers this condition does not hold true precisely. However, it is usually acceptable for general rainfall and snowmelt floods in which discharge changes slowly with time. For such floods, a person standing on the bank of a stream during a flood would most likely not perceive the vertical movement or curvature of the water surface.

*b. Gradually varied flow.* The depth and velocity change gradually along the length of the watercourse. These conditions are valid for most river flows, including floods, and the assumption of a hydrostatic pressure distribution (associated with gradually varied flow) is reasonable as long as the flow changes are gradual enough so that the imaginary lines of flow are approximately parallel.

*c. One-dimensional flow.* Variation of flow characteristics other than in the direction of the main axis of flow may be neglected and a single elevation represents the water surface of a cross section perpendicular to the flow. Thus, velocities in directions other than the direction of the main axis of flow and effects due to centrifugal force at curves, are not computed. A correction factor is applied to account for the horizontal velocity distribution.

*d. Small channel slope.* The stream channel must have a slope of 1 in 10 or less. Small slopes are required because of the assumption that the hydrostatic pressure distribution is computed from the depth of water measured vertically. For a bed slope of 1:10, which is steep for a natural channel, measuring the depth vertically results in an error of only one percent. Most floodplain studies are performed on streams that meet this requirement.

*e. Rigid boundary.* The flow cross section does not change shape or roughness during the flood. While this assumption is generally used, many alluvial streams may undergo considerable change in the shape of the bed and banks during a major event.

*f. Constant (averaged) friction slope between adjacent cross sections.* Approximation of the friction loss between cross sections can be obtained by multiplying a representative friction slope by the reach length that separates them. Various approximating equations are used to determine the friction slope. For example, in HEC-2 four equations are available, designated as average conveyance, average friction slope, geometric mean friction slope, and harmonic mean friction slope (U.S. Army Corps of Engineers 1990b). This assumption requires that cross section spacing and the selection of an appropriate friction-slope equation for computing the loss be governed by conditions in the reach.

## 6-3. Standard-step Solution

In open channel flow, the potential energy, $Z$, is specified as the height of the solid boundary confining the flow above some datum. If the pressure distribution is hydrostatic, the pressure energy, $P/\gamma$, is the depth of water above the solid boundary. These two energy terms can be added to obtain

$$WS = P/\gamma + Z \tag{6-1}$$

where $WS$ is the water surface elevation above the datum, as shown in Figure 6-1. The equation can then be rewritten

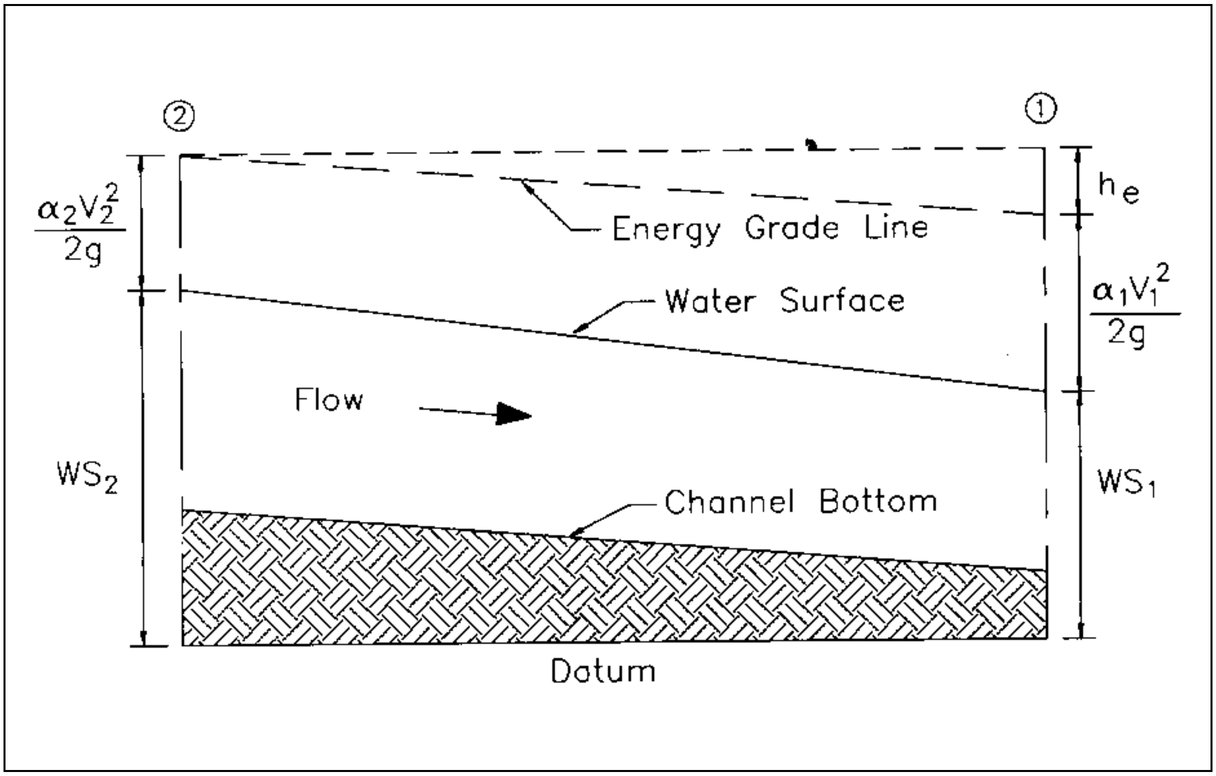


**Figure 6-1. Open channel energy relationships**

$$WS_2 + \frac{\alpha_2 V_2^2}{2g} = WS_1 + \frac{\alpha_1 V_1^2}{2g} + h_e \tag{6-2}$$

An equation for the energy head loss $h_e$ can be written as follows

$$h_e = L\bar{S}_f + C\left|\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g}\right| \tag{6-3}$$

where

$L$ = discharge weighted reach length
$\bar{S}_f$ = representative friction slope for reach
C = expansion or contraction loss coefficient

The solution of Equation 6-2 is the basis of water surface profile computations in programs such as HEC-2. The standard step method used to obtain a solution requires successive approximations. A trial value of $WS_2$ in Equation 6-2 is assumed, and values for $h_e$ and change in velocity head are computed and summed to obtain $\Delta WS$. This value is added to the known downstream water surface elevation to compute $WS_2$. The difference between trial and computed values converges with successive trials. The steps in this procedure are as follows:

*a*. Assume a water surface elevation at the upstream cross section (or downstream cross section if a supercritical profile is being calculated).

*b*. Based on the assumed water surface elevation, determine the corresponding total conveyance and velocity head.

*c*. With values from step 2, compute $\bar{S}_f$ and solve Equation 6-2 for $h_e$.

*d*. With values from steps 2 and 3, solve Equation 6-2 for $WS_2$.

*e*. Compare the computed value of $WS_2$ with the values assumed in step 1; repeat steps 1 through 5 until the values agree to within .01 feet (or .01 meters).

## 6-4. Range of Applicability

The assumptions of the method as described in section 6-2 are the basis for determining applicability. Their effects in modeling are as follows:

*a. Steady flow.* This assumption generally is not a significant problem. For most naturally occurring floods on major streams, flow changes slowly enough with time that steady flow is a fair assumption. Even when it is not, the assumption would seldom cause any computational problems. Three conditions under which a steady-flow model may not be applicable are:

(1) A rapidly moving flood wave, as from a dam breach, for which the time-dependent term of the full unsteady-flow Equation has a significant effect.

(2) Backwater effects from a downstream boundary condition, such as a tidal flow, are significant.

(3) A flat channel slope resulting in a pronounced loop effect in the relationship between discharge and elevation. See Chapter 5 for more information.

*b. Gradually varied flow.* This is a reasonable assumption in most river reaches that are free of structures and severe changes in channel geometry; however, this may not be a valid assumption in the vicinity of structures such as bridges and channel controls. The estimation of energy losses and the computation of water surface elevations in rapidly changing flow become more uncertain. Under these conditions, the estimated energy loss may be too high or too low, or the computational process may not be able to determine a water surface elevation based on computed energy losses, and a critical depth is assumed. For most floodplain studies, the critical depth solution is not valid. A critical depth solution at a cross section will not provide a basis for computing a floodway encroachment based on a change of water surface elevation.

*c. One-dimensional flow.* This may not always be a valid assumption. Two major problems that violate the assumption of one-dimensional flow are multiple water surface elevations and flow in multiple directions.

(1) Multiple water surface elevations within one cross section usually result from multiple flow paths. When the flow in each path is physically separated from the other paths, the distribution of flow in each path is a function of the conveyance (or energy loss) through the length of that path. Because the one-dimensional model distributes flow in each cross section based on the conveyance in that cross section, the flow distribution in the model is free to shift from cross section to cross section in the computational process. The traditional solution to the problem is to divide the model into the separate flow paths and compute a profile for each (see Chow 1959, Sec. 11-9).

(2) Flow in multiple directions cannot easily be modeled with a single cross section perpendicular to the flow. In cases where the flow is gradually expanding, contracting, or bending, a cross section generally can be defined that will reasonably meet the requirement, but it does take special care. When flow takes a separate path, as in the case of a levee overflow or a side diversion, the flow lost from the main channel must be separately estimated and subtracted from the main channel flow. The HEC-2 program has a split flow option to compute lateral flow losses and the resulting profile in the main channel (U.S. Army Corps of Engineers 1982a).

*d. Small channel slope.* This condition is common in natural streams. A slope less than 1 in 10 means that the pressure correction factor is close to 1 and not required. Also, the depth of flow is essentially the same whether measured vertically or perpendicular to the channel bottom (Chow 1959). For most valley streams where floodway computations are performed, a 1 in 10 slope would be considered steep. Channel slopes are usually less than 1 in 100.

*e. Rigid boundaries.* This assumption requires that the channel shape and alignment be considered constant for the period of analysis. The concern is not with long term changing boundaries, like those on meandering rivers, but with local scour and deposition that can occur in a stream during a flood event. The problem is more pronounced at major contractions, such as bridge crossings, because there is an increase in velocity with the potential for increased scour. Guidelines for determining critical scour velocities can be found in design criteria for stable channels of EM 1110-2-1610.

## 6-5. Example of Steady Flow Water Surface Profile Study

*a. Study objective.* The overall objective was a comprehensive reanalysis of water surface profiles for a reach of the Tug Fork River in the Williamson, West Virginia, flood protection project area (Williams 1988a, 1988c).

*b. Description of the study reach.* The Tug Fork River originates in the southern part of West Virginia and flows for 155 miles in a northeasterly direction to Louisa, Kentucky, where it joins the Big Sandy River.

(1) In the headwater regions the terrain is mountainous, but in the lower reaches, the valleys are wide and the hills gentle and rounded. Through most of the area, the river flows in deep, narrow, sinuous valleys between steep side ridges. Williamson is located in the lower third of the Tug River Basin, where the valley is 800 to 900 feet wide.

(2) The original water surface profile study reach extended from Kermit, West Virginia, to the central business district of Williamson, a distance of 20 miles. The general slope of this reach is about 2 feet per mile.

(3) The channel is alluvial with a bottom width of about 150 feet and stable banks with heights ranging up to 25 feet above low water. Bed sediments are sand and gravel. Vegetation, predominately conifer, lines both banks and covers the floodplain except where cleared for agricultural or industrial use.

*c. Summary of water surface profile model and parameter evaluations.* Refinements to the original HEC-2 data file included substituting field data at bridges, developing reach lengths, and assigning Manning's roughness coefficients by vegetation and land use. Channel bank limits were reestablished to better approximate the limits of bank vegetation.

(1) Sensitivity of calculated profiles was evaluated to determine the significant hydraulic parameters. Superelevation, bed scour during floods, local inflows, overbank flows, relative roughness, and seasonal vegetation roughness were analyzed. Key sources of field data for these evaluations were high-water marks from 1984 and 1977 floods and USGS gage records at Williamson.

(2) Some of the results from these evaluations were bed scour during these events was found to be negligible, superelevation did not impact except to indicate that the calibration tolerance should be relaxed from 0.5 foot to 1 foot, and local inflow changes improved agreement between calculated and observed profiles between gages.

(3) The three most significant hydraulic parameters were the identification of significant overbank flow through the town of Williamson, changes in the values of roughness as rare flood events overtopped all trees, and seasonal changes in vegetative roughness.

(4) The maximum discharge during the 1977 event was so significant that two extrapolations were made, one for a 94,000 cfs event and one for a 117,000 cfs event. The procedure for extrapolating the rating curves followed EM 1110-2-1601 which utilizes "relative roughness" and uses observed data to calculate roughness height. The details of the extrapolation procedure and other details of the study are presented in Williams (1988a, 1988c). Calibration of the HEC-2 model to the two flood events is discussed in a later section under the heading "Model Calibration and Verification" (6-11).

*Section II*
*Data Requirements*

## 6-6. Introduction to Data Requirements

The time and effort required for completion of water surface profile studies depend upon the detail of the analysis required to secure the results desired. In some cases the character of available basic data and the time available impose practical limitations on the scope of the study. In preliminary investigations a rapid approximate method may give results fully as satisfactory for the purpose involved as a more accurate but time consuming computational procedure. In other cases, the utmost degree of accuracy possible by a detailed and thorough analysis may be profitable and essential for reliable engineering. Accordingly, profile computations should be initiated with a careful appraisal of the degree of accuracy necessary for satisfactory results, considering the purpose and character of the investigations involved, the detail and probable accuracy of basic data available, the complexity of flow conditions in the stream, and the budget and time limit for completion of the studies.

*a. Theory.* Hydraulic theory is well established for channels with rigid boundaries, and computer simulation models based on this theory produce consistent and accurate results if properly applied. Major sources of error are inaccuracies in data and improper modeling of flow conditions.

*b. Categories of data.* Basic data are grouped into five categories: cross sections, reach lengths, loss coefficients, flow regime, and starting condition. The accuracy required for this data depends upon the accuracy needed in the final results. At times, it seems most economical to compensate for inadequacy of data by using safety factors such as providing liberal amounts of freeboard. In rural areas such procedures may be acceptable, but in urban areas both property damage and loss of life can result from designs based on inadequate and inaccurate data. Cross-sectional data and loss coefficients are discussed in Appendix D.

## 6-7. Flow Regime

Water surface profile computations begin at a cross section with known or assumed starting conditions and proceed upstream for subcritical flow or downstream for supercritical flow. Subcritical profiles computed by a program such as HEC-2 are constrained to critical depth or above, and supercritical profiles are constrained to critical depth and below. The program will not allow profile computations to cross critical depth except for certain bridge-analysis problems. When flow passes from one flow regime to the other, it is necessary to compute the profile twice, alternately assuming subcritical and supercritical flow (U.S. Army Corps of Engineers 1990b).

## 6-8. Starting Conditions

If feasible, profile computations should be started at a point of control where the water surface elevation can be definitely determined. This may be at a gaging station, a dam, or a section where flow is at critical depth. However, for practical reasons, it is often necessary to start the computations at other locations.

*a. Known elevation.* When a profile computation begins at a dam or a gaging station on a river where the water-surface elevation versus discharge relationship is known and is applicable to the conditions for which a profile is desired, the starting elevation can be determined from a rating curve. A common situation of this type involves the computation of a water surface profile starting at a full-pool elevation of a reservoir with a specified discharge through or over the dam.

*b. Critical depth.* In certain instances it may be feasible to start computations from a point where it is known that critical depth will occur. Critical depth in rivers may occur where the channel slope steepens abruptly, or at a natural constriction in the channel. Critical depth may be produced artificially by structures that raise the channel bottom or constrict the channel width. If a critical depth location can be determined, the critical depth option for determining the starting elevation can be specified in input to a program like HEC-2, and it will compute the critical depth and use it.

*c. Uniform flow.* If the assumption of uniform flow is reasonable, the slope-area method may be used to find a starting elevation based on the computation of normal depth. If an estimate of the slope of the energy grade line and an initial estimate of the starting water surface elevation are input to HEC-2 at a given cross section, the program will do a normal-depth calculation automatically. It will compute the discharge for the initial conditions, and compare it with the given discharge. If there is a significant difference, it will adjust the depth and repeat the computation in a series of iterations until a 1 percent difference criterion is met for the computed and given discharges.

*d. Estimated slope.* When the starting elevation for a selected discharge cannot be determined readily, it is necessary to derive a starting elevation using available expedients. One method is to select a water-surface slope on a similar stream(s), and solve Manning's Equation by trial-and-error or graphically for the water-surface elevation necessary to give that slope.

*e. Estimated stage.* Another method is to begin profile computations using a trial starting elevation at a location some distance downstream from the reach for which the backwater curve is desired. The error resulting from an incorrectly assumed trial starting elevation will tend to diminish as the computation progresses upstream. The distance downstream can be estimated from the regression equations presented in "Accuracy of Computer Water Surface Profiles" (U.S. Army Corps of Engineers 1986). Equations are presented for both critical and normal depth starting assumptions. The impact of the starting depth assumption can be tested by computing a second profile beginning at the same downstream location but at a different trial starting elevation. The starting assumption is reasonable if the two corresponding backwater curves merge into one before the computations have progressed to the reach for which the backwater curve is desired. In selecting the trial starting elevations, one elevation should be below and the other above the true elevation.

*f. Tidal conditions.* When the profile computation begins at the outlet of a stream influenced by tidal fluctuations, the maximum predicted high tide, including wind-wave set up, is taken as the starting elevation at a station usually located at the mouth of the stream.

*Section III*
*Model Development*

## 6-9. Data Sources

Data requirements for water surface profile computations were discussed in the preceding section. To reiterate, the following data are required: discharge, flow regime, starting water surface elevation, roughness and other energy loss coefficients, and the geometric data--cross sections and reach lengths. Sources for geometric data and energy loss coefficients are discussed in Appendix D. Sources for the remaining items are discussed here.

*a. Discharge.* The discharge used in water surface profile computations is generally the peak discharge associated with a given frequency. For example, in a multiple-profile analysis for a floodplain-information study, peak discharges for the 10-, 50-, 100-, and 500-year events may be required. Peak discharges are generally obtained from flood-frequency analysis or from the application of historical or design storm precipitation data to rainfall-runoff models such as HEC-1.

*b. Flow regime.* Since water surface profile computations in a model such as HEC-2 do not cross critical depth, it is necessary at the outset of an analysis to decide whether to analyze the flow as subcritical or supercritical. The flow regime is subcritical in most rivers; however, if this assumption is used and is incorrect, program output will indicate that a wrong decision may have been made. Critical depth will be assumed and noted in the output for cross sections in the model where the regime is different from that assumed. For reaches in which flow actually passes from one regime to the other, it may be necessary to make a separate computation for each regime and combine the results for a complete analysis.

*c. Starting water surface elevation.* Alternative methods for determining the starting water surface elevation are discussed in the preceding section on data requirements.

## 6-10. Data and Profile Accuracy

It would seem, from the list of suggested cross-section locations in Appendix D, that the effects of most undesirable features of nonuniform, natural stream channels can be lessened by taking more cross sections. While this is generally true, time, cost, and effort to locate and survey cross sections must also be considered. A balance must be set between the desirable number of cross sections and the number that is practical. Accuracy of the data and the profiles should be part of the balance consideration.

*a. Associated error.* Errors associated with computing water surface profiles with the step-profile method can be classified as basic theory, computational, or data estimation (McBean and Pernel 1984).

(1) Minimizing error in the application of theory is the responsibility of the engineer conducting the study.

(2) Computation errors include numerical round-off and numerical solution errors. The former is negligible using today's modern computers and the latter can be minimized by employing readily available mathematical solution techniques.

(3) Data estimation errors may result from incomplete or inaccurate data collection and inaccurate data estimation. The sources of data estimation errors are the accuracy of the stream geometry and the accuracy of the method used and data needed for energy loss calculations. The accuracy in stream geometry as it affects accuracy of computed profiles is important. The accuracy of energy loss calculations depends on the validity of the energy loss Equation employed and the accuracy of the energy loss coefficients. The Manning Equation is the most commonly used open channel flow Equation and the coefficient measuring boundary friction is Manning's *n*-value.

*b. Accuracy of data collection and estimation.*

(1) Aerial survey and topographic map accuracy. Stream cross-sectional geometry obtained from aerial surveys (aerial spot elevations and topographic maps) that conform to mapping industry standards are more accurate than is often recognized. Cross-sectional geometry obtained from aerial spot elevation surveys is twice as accurate as cross-sectional geometry obtained from topographic maps derived from aerial surveys for the same contour interval.

(2) Profile accuracy prediction. The effect of aerial spot elevation survey or topographic mapping accuracy on the accuracy of computed water surface profiles can be predicted using the mapping industry accuracy standards, reliability of Manning's coefficient, and stream hydraulic variables.

(3) Manning's coefficient estimates. The reliability of the estimation of Manning's coefficient has a major impact on the accuracy of the computed water surface profile. Significant effort should be devoted to determining appropriate Manning's coefficients.

(4) Additional calculation steps. Significant computational errors can result from using cross-sectional spacings that are often considered to be adequate. The

errors are due to inaccurate integration of the energy loss-distance relationship that is the basis for profile computations. This error can be effectively eliminated by adding interpolated cross sections (more calculation steps) between surveyed sections.

(5) Aerial survey procedures. Aerial spot elevation survey methods are more cost effective than field surveys when more than 15 survey cross sections are required. Use of aerial spot elevation survey technology permits additional coordinate points and cross sections to be obtained at small incremental cost. The coordinate points may be formatted for direct input to commonly used water surface profile computation computer programs.

*c. Errors in the data.*

(1) Profile errors resulting from use of commonly applied field survey methods of obtaining cross-sectional coordinate data are a function only of Manning's coefficient of reliability "Nr" (U.S. Army Corps of Engineers 1986). Computed profile error resulting from survey error is small even for rough estimates of Manning's coefficient.

(2) Profile errors resulting from use of aerial spot elevation surveys for obtaining cross-sectional coordinate data vary with the contour interval and reliability of Manning's *n*-value.

(3) The small profile error for the aerial spot elevation survey method is due to the high accuracy of aerial spot elevation surveys and the randomness of the measurement errors at the individual coordinate points. The latter results in compensating errors along the cross-sectional alignment. For the error prediction determined from the regression Equations to be valid, eight or more cross-sectional coordinate points are needed to ensure that the randomness and thus compensatory error process has occurred.

(4) The error in computed water surface profiles increases significantly with decreased reliability of Manning's coefficient. The profile errors resulting from less reliable estimates of Manning's coefficient are several times those resulting from survey measurement errors alone.

(5) There is significantly greater error for larger contour intervals for topographic maps than for aerial spot elevation surveys. Data from topographic maps are simply less accurate. Also, topographic map cross-sectional elevations can only be obtained at the contour intervals. Significant mean profile errors (greater than 2 feet) may be expected for analyses involving steep streams, large contour intervals, and unreliable estimates of Manning's coefficients.

(6) The error prediction Equations in "Accuracy of Computed Water Surface Profiles" (U.S. Army Corps of Engineers 1986) may be used to determine the mapping required to achieve a desired computed profile accuracy.

## 6-11. Model Calibration and Verification

*a. Calibration.* The goal of calibration is to obtain a set of parameters for a model so that it will respond like the physical system it represents. A calibrated steady-flow water surface profile model should compute water surface elevations that are essentially the same as observed elevations (from high water marks or gage readings) not only for the set of conditions used in calibration but for others as well. This is accomplished with a trial-and-error procedure in which a water surface profile is computed with an initial set of parameters and compared to the observed data. The parameters are adjusted on the basis of the comparison, and the procedure is repeated until a suitable fit is obtained.

*b. Verification.* Verification is closely akin to calibration in that it, too, amounts to the comparison of computed model output to observed data. The distinction between the two procedures is usually made on the basis of timing and the different data sets involved. A model is first calibrated to one set of observed data and then verified with another set.

*c. Factors in reconciling differences.* Several factors that might be considered in reconciling differences between computed and observed data (Hoggan 1989) are as follows:

(1) There is usually some leeway in assigning *n* values, and these might be adjusted upward or downward slightly to achieve a better fit of computed and observed data.

(2) The reliability of the discharge values from a hydrologic model or other sources might be questioned. If differences in computed and observed profiles are great (a few feet or more), erroneous discharge values might be the problem, and this possibility should be investigated.

(3) Even though the precision of survey data is usually not a problem (as discussed in 6-10*c*), major

errors in survey data can occur, having significant impact on the accuracy of computed profiles, and may warrant checking.

(4) At some locations changing the bridge method used in the model may improve the computed profile.

(5) If a high water mark is unusually high at a bridge, it may have resulted from a snag or debris caught on the piers. A dam failure or diversion upstream can also abnormally affect high water marks.

(6) The replacement of a bridge, channel modifications, construction of encroachments, and development of adjacent land since the water marks were made would complicate calibration and verification.

(7) Questionable data are always a possibility. For example, inaccurate rainfall data could cause discharge values to be off, and information from local residents regarding high water marks may be in error.

*d. Other considerations.* Other considerations for the evaluation of the high water marks (Williams 1988b) are as follows:

(1) Looped rating curves. Some rivers exhibit a looped rating curve which indicates that for a given depth the discharge will be greater on the rising stage of a flood than on the falling stage. This leads to the maximum water surface elevation not corresponding to the peak discharge, and can result in calibrating a model to high water marks that are not consistent with the given discharge.

(2) Superelevation. Sometimes high water marks are taken at curves on a river in which the water surface is superelevated at the outside of a bend. Because a one-dimensional steady-flow model assumes a horizontal water surface, the computed elevation must be adjusted for this superelevation before it is compared with high water marks.

(3) Waves and "set up". If a debris line is used to determine high water marks, it may be higher than the actual water surface elevation because of the effect of waves. Errors can occur from water-momentum changes which result in a "set up" of the water surface elevation. This may occur if the debris line is not parallel to the flow, if the flow must make an abrupt change in direction, or at "dead end" areas.

(4) Backwater areas. If water surface elevations are affected by backwater, high water marks will be higher than normal-depth elevations. The effects of the backwater can be determined by varying the downstream control in the model. By using the downstream elevations required to match the high water marks, it can be determined if these elevations are within the expected downstream elevation range. This problem usually arises for a study reach on a tributary at a location near the confluence of the tributary with the main stream. If channel modifications on the tributary affect the downstream control, the calibrated *n* value for a given discharge may no longer be valid.

*e. Adjusting n.* Several suggestions for adjusting *n* values in the calibration process (Williams 1988a, 1988c) are as follows:

(1) Flow resistance caused by vegetation can vary due to the depth of flow, vegetative stand characteristics (see Figure 6-2), and amount of foliage. Differences in seasonal foliage may need to be considered when calibrating events that occur at different times of the year.

(2) Flow resistance is affected by bedforms and surface (or grain) resistance. Simons and Richardson (1966) describe the types of bedforms and their relative resistance (Figure 6-3). Brownlie (1981) has developed a flow resistance relationship which takes into account both the surface and the bedform. This should be used only in the alluvial portion of a river.

(3) A compound channel is one with laterally varying roughness and flow depth, as depicted in Figure 6-4. If compound channel subsections influence each other's flow by phenomenon such as momentum exchange between subsections, a composite *n* is recommended because each subsectional roughness height does not change appreciably with flow depth, but the composite height does (and so does the composite *n*). See EM 1110-2-1601, Appendix IV for details.

(4) The assignment of *n* values in water surface profile modeling should be done in a systematic and defensible manner by identifying the types of roughness encountered in the prototype along with a corresponding range of assigned *n* values. The reaches are then categorized by types of roughness and assigned *n* values within the established range. If this is done early in a study, it can be of value in establishing a good initial model and

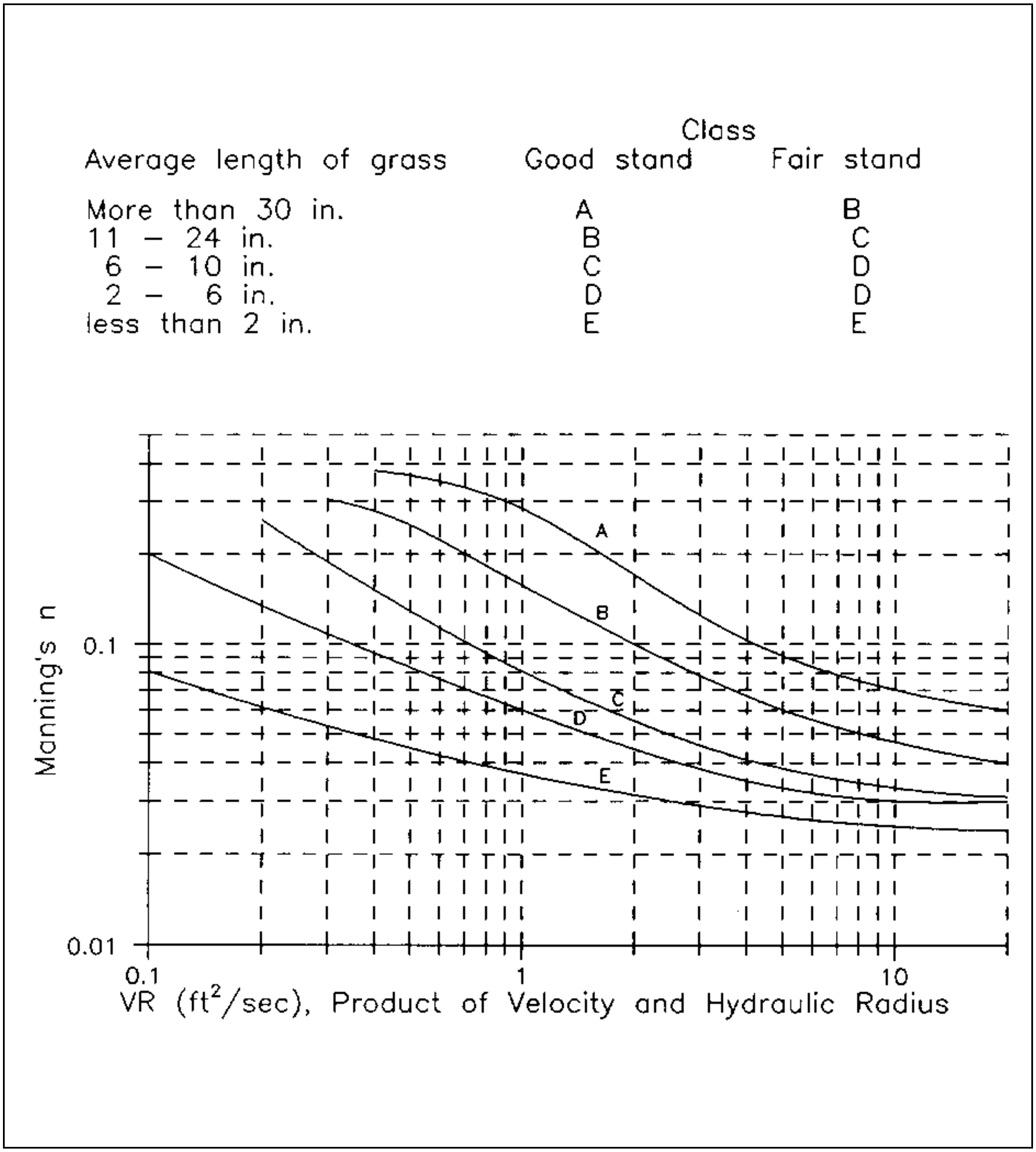


**Figure 6-2. The behavior of Manning's *n* in grassed channels**

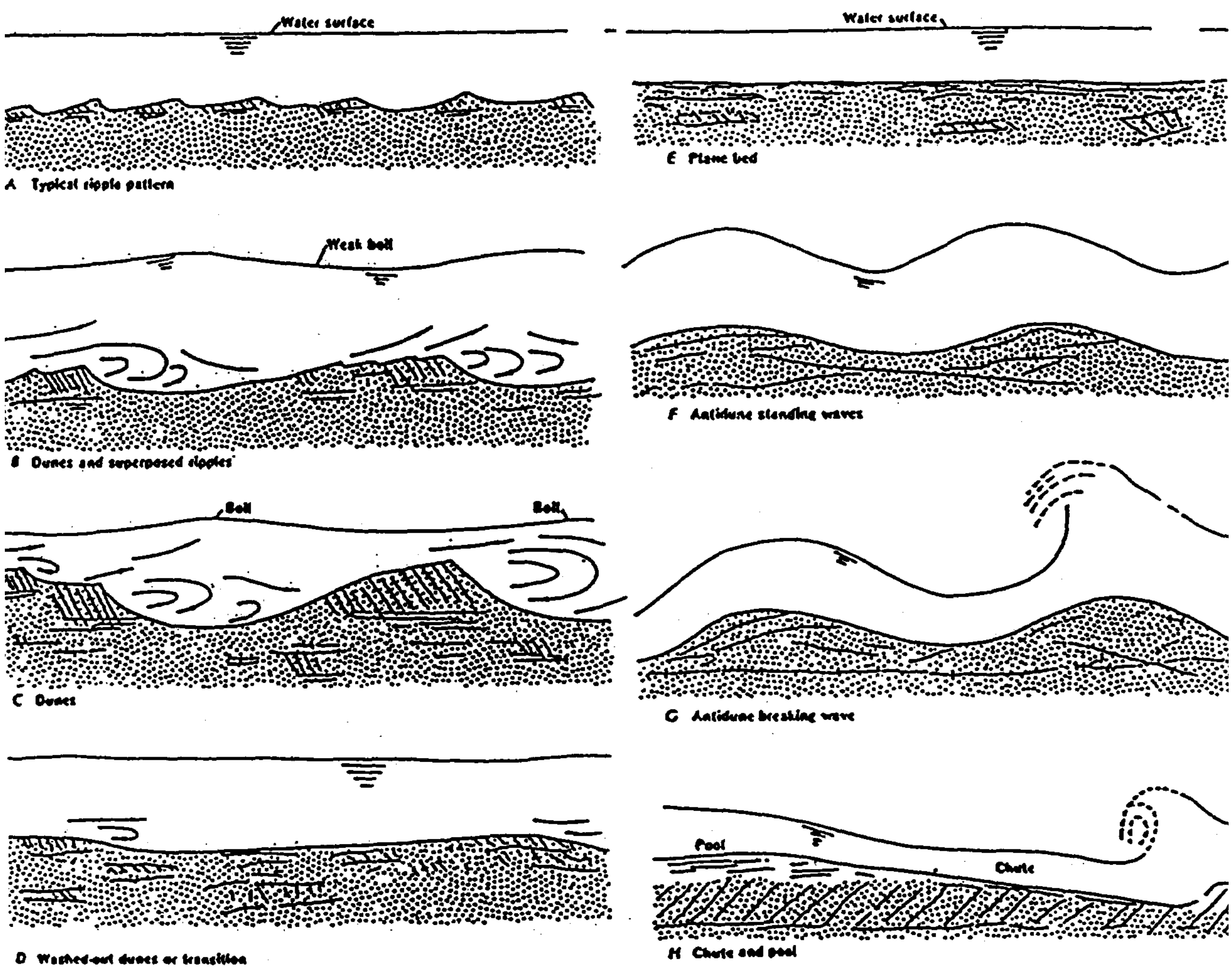


Figure 6-3. Types of bed forms and their relative resistance to flow

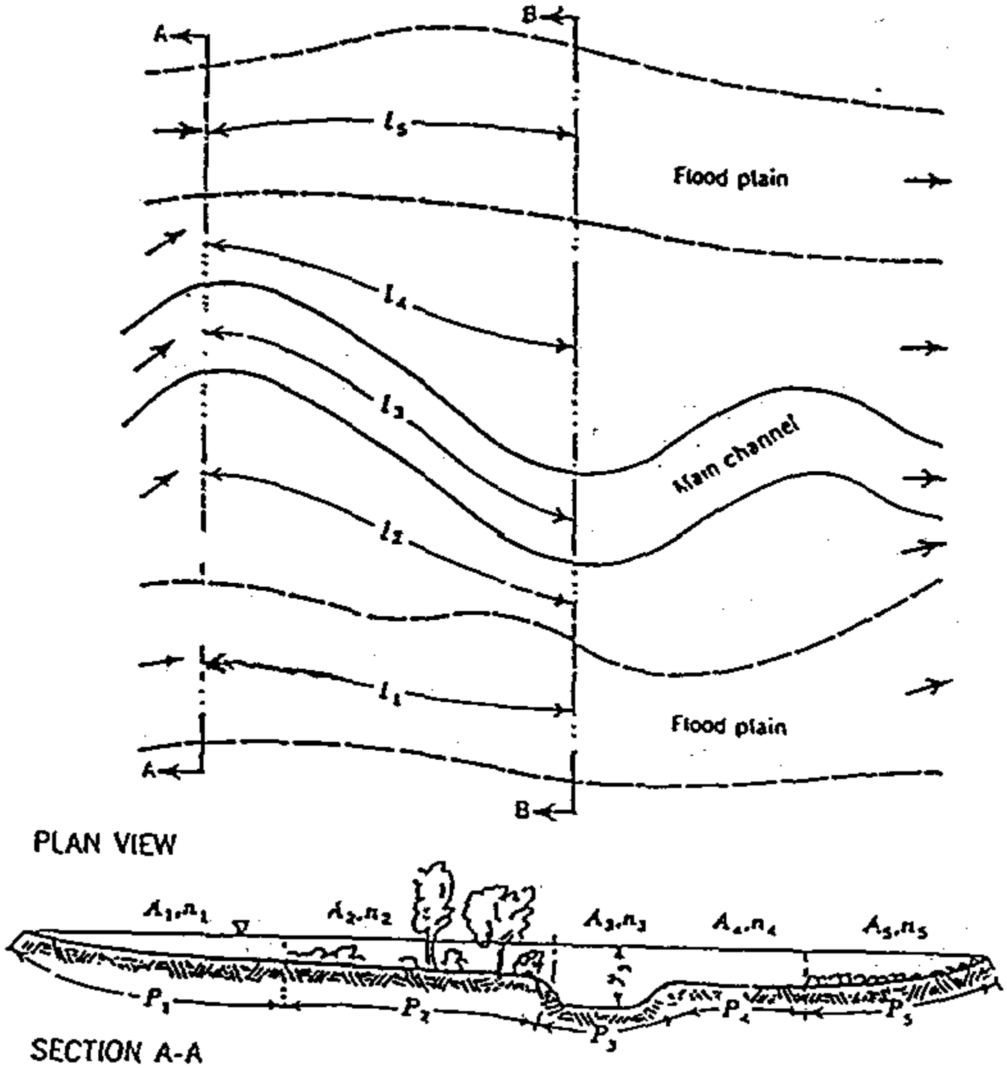


**Figure 6-4. Compound channel with laterally varying Roughness and flow depth**

become the basis for determining *n*-adjustment limits. An example of a table of *n* values used for model calibration in the Williamson, West Virginia, flood control project is presented in Table 6-1.

*f. Example of HEC-2 calibration.* A brief description of the calibration of an HEC-2 model used on 20-mile reach of the Tug River in West Virginia is presented in this section. The model was calibrated to floods that occurred in 1984 and 1977. For additional detail on the calibration, see Williams (1988a, 1988c).

(1) Using Chow (1959) as a guide, Manning's *n*-values were assigned to specific reaches of the river and put in the HEC-2 model. The initial *n*-values were adjusted to reproduce observed high water marks. These marks were reproduced within 0.5 foot except for three marks that were reproduced within 1.0 foot, attributable to superelevation "runup" at bends.

(2) Due to inconsistencies in the observed water-surface profiles for the 1984 flood, adjustments to the initial tributary discharges were made after the rainfall data were reexamined and the 1984 flood reconstituted. This changed the main stem discharge at the Kermit gage from 82,000 to 58,000 cfs for the 1984 flood.

(3) The calibration of the 1984 flood resulted in a channel Manning's *n* of 0.058 at the USGS gage in Williamson. The 1977 flood calibration produced channel *n*-values of 0.041 and 0.028 for the 94,000 and 117,000 cfs calibrations, respectively.

(4) Analyses of the detailed USGS discharge/velocity measurements from the 1984 flood indicated that significant flow through the Williamson central business district (CBD) occurred during the 1977 flood. To simulate this, the HEC-2 model was adjusted to reflect the geometry of the buildings and streets, and this overbank area was assigned a Manning's *n*-value of 0.020. Checks were made to assure that side flow over the existing floodwall was sufficient to meet the CBD conveyance potential.

**Table 6-1**
**Roughness Description and Manning *n* Values for 1977 Flood Calibration**

| Reach River Miles | Left Overbank Description | n | Channel Description[a] | n | Right Overbank Description | |
|---|---|---|---|---|---|---|
| 38.4-43.86 | Clearings of Grassy and Developed Areas to Light to Medium Dense Brush | 0.01 to 0.069 Avg 0.057 | Gradual Bendways; Typical Side Slopes of Light to Medium Dense Brush with Reaches of Medium Dense Brush | 0.036 | Clearings with Scattered Development and Road and Railroad ROW's | 0.041 to 0.069 Avg 0.044 |
| 43.86-49.07 | Generally Clearings of Grassy and Developed Areas with Intermediate Reaches of Light to Medium Dense Brush | 0.041 to 0.069 Avg 0.045 | Increased Sinuosity; Sharper Bendways; Typical Side Slopes of Medium to Heavy Dense Brush with Reaches of Light to Medium Dense Brush | 0.041 | Clearings with Reach of Development and Scattered Vegetation; Grassy Reach and a Reach of Light to Medium Dense Brush | 0.041 to 0.048 Avg 0.045 |
| 49.07-53.86 | Grassy Clearings with Some Development; Reaches of Light to Medium Dense Brush | 0.044 to 0.051 Avg 0.045 | Gradual Bendways; Typical Side Slopes of Light to Medium Dense Brush; A Reach of Medium to Dense Brush | 0.036 | Developed with Short Reaches of Grassy to Light to Medium Dense Brush | 0.041 to 0.048 Avg 0.045 |

*Section IV*
*Special Problems*

## 6-12. Introduction to Special Problems

The nature of flow profiles and energy losses at natural or constructed channel features that cause increased energy losses or modified boundary conditions are discussed. Special modeling approaches are presented for various kinds of problems.

## 6-13. Bridge Hydraulics

*a. Nature of flow through a bridge constriction.* Flow through a bridge in a wide floodplain has been conceptualized as having four regions: accretion, contraction, expansion, and abstraction (Laursen 1970).

(1) The region of accretion begins upstream from the bridge, a distance just far enough so that the flow is not constricted by the influence of the bridge and the streamlines are parallel. This region extends downstream to a point close to the upstream face of the bridge. As the flow moves through this region towards the bridge, the flow in the overbanks of the floodplain must move laterally toward the channel so that it can pass through the bridge opening. Since the contraction takes place over a considerable distance, the type of flow is "gradually varied."

(2) The region of contraction begins immediately above the upstream face of the bridge where the first region ends and extends through the bridge. The flow contracts more severely in this region to pass through the bridge opening, and the geometry of the opening has a significant effect on the amount of energy loss. A jet is generally formed in the bridge opening, and extends into the region of expansion immediately downstream from the bridge, where it expands through turbulent diffusion and mixing. The type of flow is "rapidly varied" in these two regions of severe contraction and expansion, and the energy losses are relatively high compared to the other two regions.

(3) The region of abstraction extends downstream from the region of expansion to a point where the flow is fully expanded within the confines of the floodplain and the streamlines are again parallel. In this region the flow is "gradually varied" as it expands laterally away from the channel to fill the floodplain.

*b. Backwater effects of bridges.* Some of the findings of extensive studies on backwater effects of bridges (Bradley 1978) are depicted in Figures 6-5 and 6-6.

(1) The bridge constriction produces practically no alteration of the shape of the streamlines near the center of the channel (Figure 6-5); however, a very marked change is in evidence near the abutments. The momentum of the flow from both overbanks (or floodplain) must force the advancing central portion of the stream over to gain entry to the constriction. After leaving the constriction the flow gradually expands (5 to 6 degrees per side) until normal conditions in the stream are reestablished.

(2) Constriction of the flow causes a loss of energy, the greater portion occurring in the expansion downstream. In a subcritical flow regime, the effect of the constriction is reflected in a rise in water surface and energy grade line upstream from the bridge. This is illustrated with the centerline profile of the stream flow shown in Figure 6-6. The normal stage of the stream without the channel constriction is represented by the dashed line labeled N.W.S. (natural water surface). The water surface as affected by the bridge constriction is represented by the solid line and labeled W.S. The water surface is above the normal stage at cross section 1 by the amount of h1*, which is referred to as "bridge backwater." The flow crosses through normal stage close to cross section 2, reaches minimum depth near cross section 3, and returns to normal stage downstream at cross section 4.

*c. Types of flow at bridges.* One of several different types of flow may exist at a bridge depending upon the regime and the flow depth relative to key elevations of the bridge and approach structures. In addition to four different classes of low flow, pressure flow, weir flow, and combinations of weir and pressure or weir and low flow are possible. A typical discharge rating curve is shown in Figure 6-7.

*d. Bridge loss calculations.* The energy losses at a bridge can be divided into two categories: those that occur in the approach reaches immediately upstream and downstream from the bridge and those that occur through the structure. In computer programs such as HEC-2, the first category is computed with standard step profile calculations that use Manning's Equation to determine friction losses and apply contraction and expansion

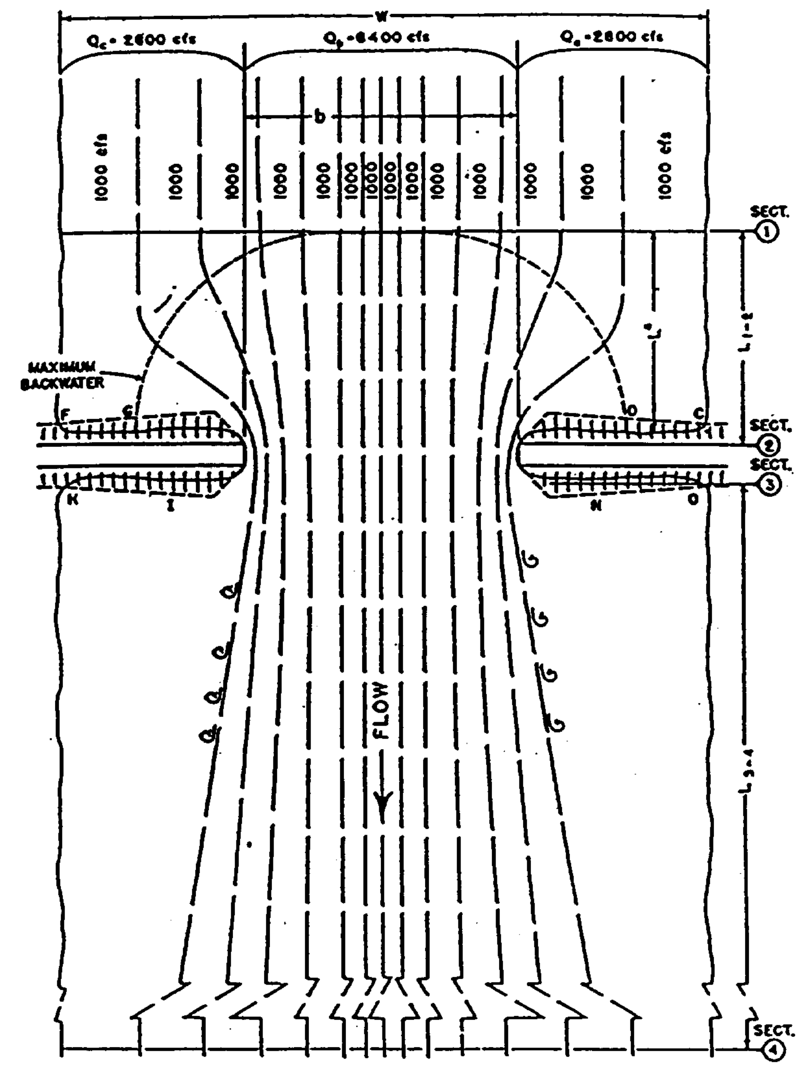


**Figure 6-5. Flow lines for typical normal bridge crossing**

coefficients to changes in velocity head between adjacent cross sections to determine other losses. The second category of losses, which occurs in the flow through the bridge structure, is determined by one of three different methods: the normal bridge method, the special bridge method, or by external hydraulic calculations input to the program. The special culvert method available for analyzing energy losses through culverts is covered in a subsequent section of this chapter.

(1) The approach reach on each side of a bridge generally requires two cross sections: one next to the face of the bridge and one at the other end of the reach. On the upstream side of the bridge, the length of the approach for contraction of the flow is usually set at a distance equal to one times the average of the two abutment projections. On the downstream side, the length of the reach for expansion is usually set at a distance of four times the average of the abutment projections. See Figure 6-8.

(2) The normal bridge method computes losses through the bridge with the standard step method in the same manner the program computes losses between natural river cross sections. Two or more additional cross sections are located within the bridge opening to define the geometry of the bridge structure and changes in

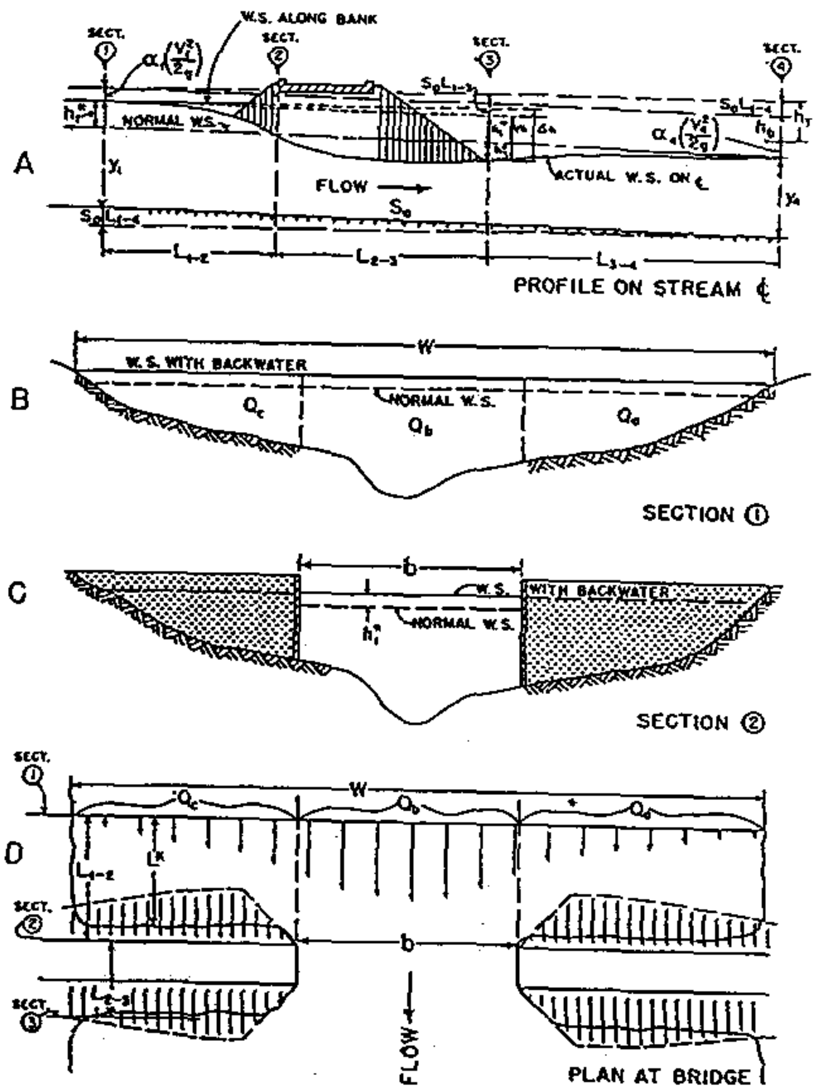


Figure 6-6. Stream profile and cross sections for normal bridge crossing, wingwall abutments

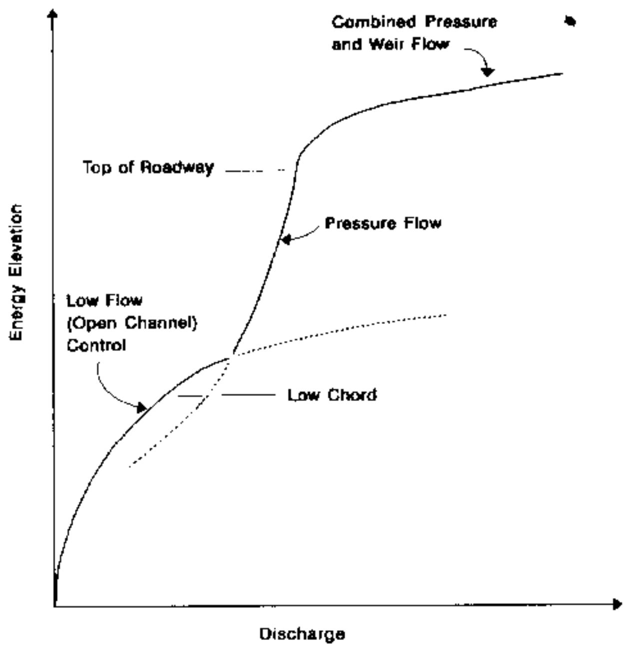


**Figure 6-7. Typical discharge rating curve for bridge culvert**

roughness for the bridge surfaces. In the computations, the area of the structure below the water surface is subtracted from the total flow area, and the wetted perimeter is increased where the water is in contact with the structure.

(3) The special bridge method computes the hydraulic losses through the bridge using hydraulic equations. The program determines whether the flow is low flow, pressure flow, weir flow, or a combination, and then applies the appropriate equations. Schematic flow diagrams and a description of the decision logic for this process, which is quite complex, are presented in the HEC-2 user's manual (U.S. Army Corps of Engineers 1990b).

(4) Externally computed bridge losses can be input to the program as computed changes in water surface elevations between cross sections located on opposite sides of the bridge.

(5) Guidelines for selecting a method for a particular bridge analysis are presented in the HEC-2 user's manual (U.S. Army Corps of Engineers 1990b). In general, the normal bridge method is most applicable when friction losses are the predominate consideration, or the conditions make it impractical to use the special bridge method. The special bridge method is most applicable for computing weir flow, pressure flow, low flow, or a combination of these that can be modeled effectively with the hydraulic equations available in the method. If the bridge acts as a hydraulic control and a rating curve is available, reading in the known water surface elevations would be the preferred method.

## 6-14. Culvert Hydraulics

*a. Culvert loss calculations.* Computation of the energy losses in the transition sections upstream and downstream from a culvert is almost the same as for a bridge. In the computation of the loss through the

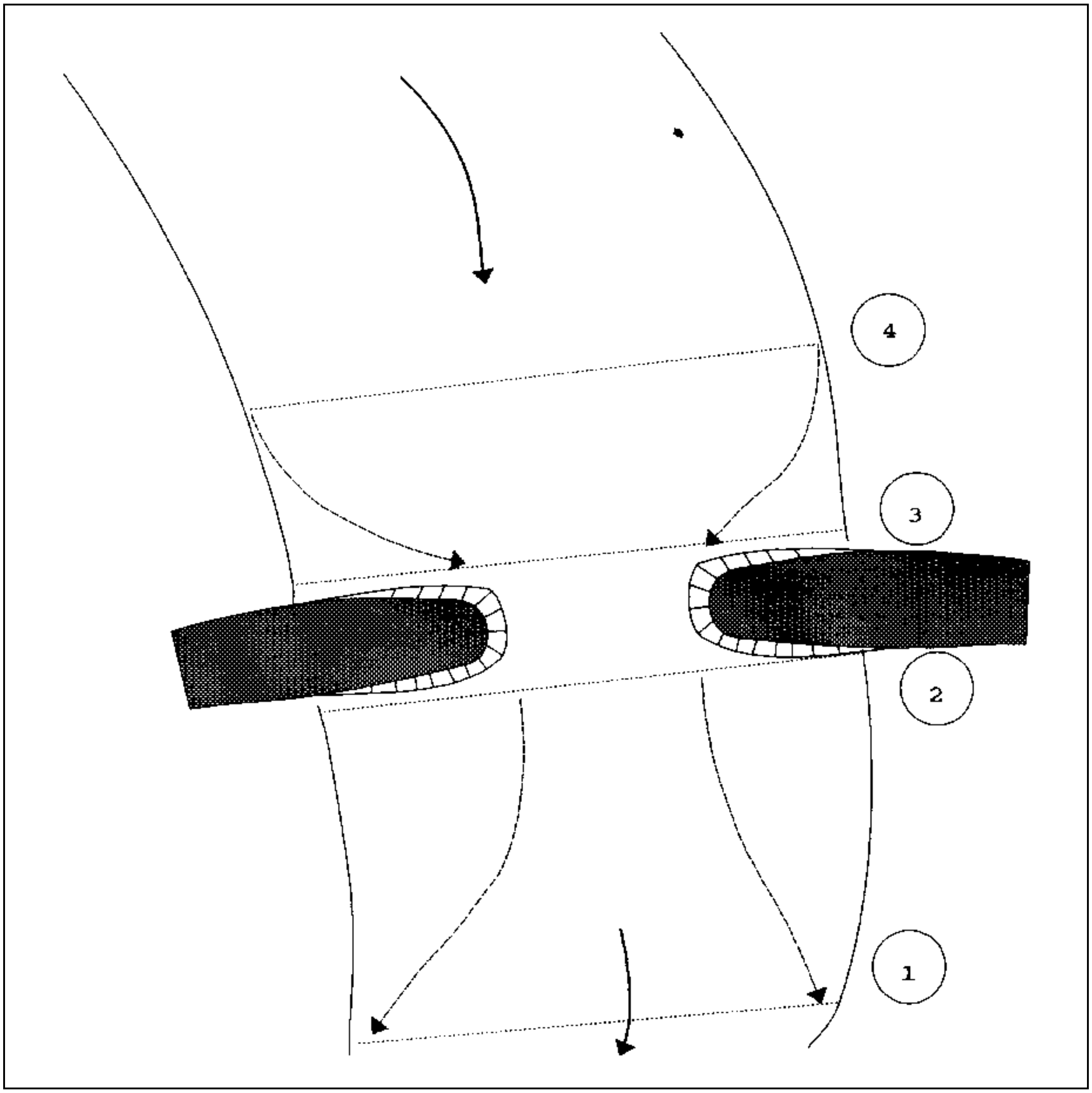


**Figure 6-8. Cross section locations in the vicinity of bridges**

culvert the concepts of "inlet control" and "outlet control" are used.

*b. Inlet and outlet control.* Inlet control of the flow occurs if the flow capacity of the culvert entrance is less than the flow capacity of the culvert barrel. Outlet control occurs if the culvert capacity is limited by downstream conditions or by the flow capacity of the culvert barrel. The headwater, which is the depth of water at the culvert entrance measured from the invert, is computed for a given flow rate under both inlet control and outlet control conditions. The higher value computed indicates which condition "controls," and it is this value that is used to determine the culvert loss.

(1) For inlet control, a series of equations that have been developed from extensive laboratory tests (U.S. Department of Transportation 1985) is used to calculate the headwater under various conditions. The headwater is computed assuming that the inlet acts as an orifice or a weir, and the capacity depends primarily on the geometry of the culvert entrance.

(2) For outlet control, the headwater is computed by taking the depth of flow at the culvert outlet, adding all head losses, and subtracting the change in the flow line (invert) elevation from the upstream to the downstream end. This is a complex process that must consider several conditions within the culvert and downstream of the culvert. A flow chart and description of the equations used in the computations are presented in the HEC-2 user's manual (U.S. Army Corps of Enigneers 1990b).

## 6-15. Limits of Effective Flow

Irregularities in the natural topography or the introduction of structures such as bridges or levees into a watercourse may require that field topographic data be modified to depict the effective flow areas through the channel irregularities or structures. Numerical models such as HEC-2 contain capabilities to restrict flow to the effective flow areas of cross sections. Among these capabilities are options to simulate sediment deposition, to confine flows to leveed channels, to block out road fills and bridge decks, and to analyze floodplain encroachments. Figure 6-9 illustrates these effective flow area modifications. In modeling it is important to study carefully the flow pattern of rivers being analyzed to determine effects of levees, bridges, and other obstructions to natural flow patterns. Appendix 4 of the HEC-2 user's manual provides guidance for modeling effective flow areas.

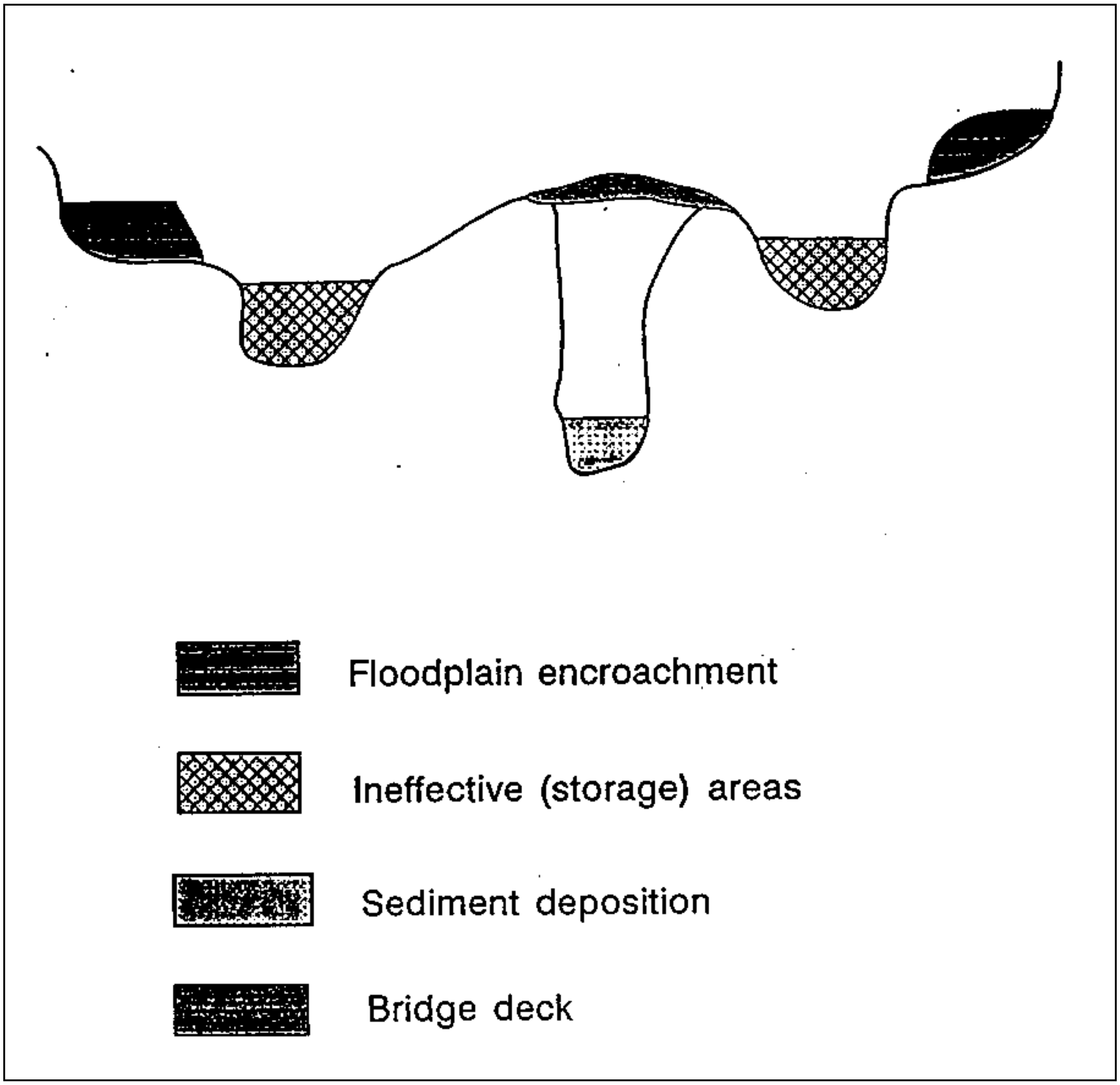


**Figure 6-9. Types of effective flow options**

## 6-16. Channel Controls

Any constriction in a channel that backs up water is a "control," and if the bed and banks of the channel at a control do not change, a constant relationship between discharge and water surface elevation will be maintained. The location of a control in a channel is called a "control section." And a control section controls the flow in such a way as to restrict the transmission of the effect of changes in flow condition either in an upstream direction or a downstream direction depending on the state of flow in the channel (Chow 1959). Streams are commonly made up of alternate reaches of slack water and rapids, and the head of a rapids being necessarily of a permanent nature is a control that tends to back water upstream.

*a. Critical depth.* The condition of critical depth implies a unique relationship between depth and discharge that can only occur at a control. The flow regime can pass from subcritical to supercritical, or vice versa, only if the flow passes through critical depth. Critical depth occurs when subcritical flow passes over a weir or free outfall. It may occur if the channel bottom is abruptly elevated or the side walls are contracted. In fact, measuring flumes are designed to force flow through critical depth by raising the bottom and narrowing the width of the channel. The discharge is determined by simply measuring the depth in the flume (Bedient and Huber 1988).

*b. Importance of controls in computing water surface profiles.* Since a control section holds a definitive stage-discharge relationship, it is a suitable location for developing discharge rating curves for water surface profile analysis. It is common practice to obtain starting water surface elevations from rating curves or conditions of critical depth at control sections. High water marks and gage readings at control sections are useful data in model calibration and verification.

## 6-17. River Confluences

*a. Confluence of a river.* At the confluence of a river and one of its tributaries, the determination of the water surface elevation of each stream immediately upstream from the confluence is necessary to continue the backwater computations up the main river or the tributary.

*b. Example.* The procedure in solving this problem at the confluence of the Missouri and Kansas Rivers is shown by example (EM 1110-2-1409) in Table 6-2. A discharge of 81,000 cfs from the Kansas River combines with 350,000 cfs from the Missouri River to give a total discharge of 431,000 cfs immediately below the confluence. Cross sections 1K and 6 are located immediately upstream from the confluence of the two streams, as shown in Figure 6-10. The hydraulic elements of cross sections 5, 6, 7 and 1K are shown in Table 6-3.

(1) The friction slope for each cross section is computed for the discharge of 81,000 cfs, at cross section 1K and 350,000 cfs at cross section 6. The friction-head loss $h_f$ is then computed, using the average friction slope from cross sections 5 to 1K on the Kansas River and from 5 to 6 on the Missouri River.

(2) The velocity head for cross section 5 is computed at a discharge of 431,000 cfs, and the velocity head for cross sections 1K and 6 is taken as the weighted average velocity head for the discharge of 431,000 cfs through the combined area of the two cross sections. The total $V^2Q$ value is determined for the combined area and divided by 431,000 to obtain the average $V^2$.

(3) The resulting change of 0.28 feet (h) between cross sections 5 and the combined area is added to the $h_f$ of 0.10 feet to obtain the total rise in water surface of 0.38 feet between cross sections 5 and 1K. Likewise, the same change is added to $h_f$ of 0.16 feet between cross sections 5 and 6 to obtain the total rise in water surface of 0.44 feet between backwater elevations.

(4) The method as described in the preceding paragraphs should be applied only to channels having low velocities not exceeding about 10 feet per second.

(5) Computer programs such as HEC-2 can compute water surface profiles for tributaries together with profiles for the main stream in a single execution of the program (U.S. Army Corps of Engineers 1990b).

## 6-18. Changing Flow Regime

*a. Steady-state water.* Most commercially available steady-state water surface profile programs such as HEC-2, can only simulate one regime of flow for a single profile computation. Whenever the calculated flow profile would cross critical depth from either the subcritical or supercritical regimes, or whenever the simulation cannot converge to a solution, critical depth at that location is assumed. For the majority of subcritical flow situations critical depth is a good assumption. However, in supercritical reaches in particular, the critical depth assumption may not be satisfactory.

**Table 6-2**
**Backwater Computations by Method 1, Missouri and Kansas Rivers at Kansas City**

| Sec. No. | River Mile | Reach L | Area A | P | R | $R^{2/3}$ | n | $K_d$ n=.01 V | $K_d$ n=.01 Q | S | Mean S | $h_f$ | V | Q | $V^2Q$ | $V^2$ ave. | $h_v$ | $h_{v_1} - h_{v_2}$ | Total H | W.S. Elev. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 |
| 1 | 377.88 | | 6,080 | 497 | 12.2 | | 0.03 | 1.80 | 8,600 | | | | 2.36 | 14,400 | 80,300 | | | | | |
| 732.2 | | | 38,000 | 910 | 42.4 | | 0.025 | 7.22 | 278,700 | | | | 10.81 | 416,600 | 48,700,000 | | | | | |
| | | | | | | | | | 286,300 | 0.000824 | | | | 431,000 | 48,780,300 | 112.2 | 1.76 | | | 732.23 |
| 2 | 377.78 | 1080 | 2,880 | 210 | 12.3 | | 0.05 | 1.86 | 4,100 | | | | 2.70 | 7,000 | 51,000 | | | | | |
| 732.5 | | | 41,600 | 1310 | 31.8 | | 0.025 | 5.97 | 348,300 | | | | 10.30 | 424,000 | 44,100,000 | | | | | |
| | | | | | | | | | 352,300 | 0.000292 | 0.000436 | 0.27 | | 431,000 | 44,151,000 | 108.4 | 1.80 | 0.27 | 0.44 | 732.66 |
| 3 | 377.04 | 845 | 1,400 | 123 | 11.4 | | 0.05 | 1.21 | 2,100 | | | | 2.13 | 3,000 | 13,800 | | | | | |
| 733.3 | | | 80,300 | 1370 | 22.0 | | 0.025 | 5.09 | 301,200 | | | | 5.31 | 428,000 | 31,800,000 | | | | | |
| | | | | | | | | | 303,300 | 0.000168 | 0.000247 | 0.21 | | 431,000 | 31,813,800 | 72.2 | 1.12 | 0.47 | 0.68 | 733.37 |
| 4 | 376.83 | 2080 | 66,400 | 2170 | 30.2 | | 0.025 | 5.77 | 377,800 | 0.000131 | 0.000167 | 0.34 | 6.60 | 431,000 | | 43.6 | 0.68 | 0.44 | 0.78 | 734.15 |
| 734.0 | | | | | | | | | | | | | | | | | | | | |
| 5 | 376.65 | 1080 | 9,040 | 754 | 12.0 | | 0.03 | 1.56 | 14,100 | | | | 1.73 | 15,700 | 47,000 | | | | | |
| 734.5 | | | 64,800 | 2100 | 30.6 | | 0.025 | 5.62 | 372,600 | | | | 6.47 | 415,300 | 17,380,000 | | | | | |
| | | | | | | | | | 387,900 | 0.000123 | 0.000127 | 0.21 | | 431,000 | 17,427,030 | 40.4 | 0.63 | 0.06 | 0.26 | 734.41 |
| | | BALANCE SECTION | | | | | 350,000 c.f.s. from Upper Missouri | | | | 81,000 c.f.s. from Kansas River | | | | | | | | | |
| IX | Mouth Kansas River | | | | | | | | | | | | | | | | | | | |
| 734.0 | | 1430 | 27,700 | 844 | 32.8 | | 0.025 | 6.08 | 168,600 | 0.000023 | 0.000073 | 0.10 | 2.93 | 81,000 | 695,000 | | | 0.38 | 0.36 | 734.79 |
| 6 | Missouri River | | | | | | | | | | | | | | | | | | | |
| 734.8 | 376.86 | 1580 | 9,300 | 735 | 12.4 | | 0.05 | 1.50 | 14,000 | | | | 1.41 | 13,200 | 26,200 | | | | | |
| | | | 84,000 | 2100 | 30.0 | | 0.025 | 5.84 | 380,600 | | | | 5.19 | 336,800 | 9,070,000 | | | | | |
| | | | | | | | | | 383,200 | 0.000076 | 0.000101 | 0.16 | | 350,000 | | | | | | |
| | | | | | | | | | | | | | | 431,000 | 9,791,200 | 22.7 | 0.35 | 0.38 | 0.44 | 734.65 |
| | Proceeding up the Missouri River | | | | | | | | | | | | | | | | | | | |
| 7 | 376.41 | 2420 | 16,400 | 614 | 26.8 | | 0.04 | 2.45 | 37,800 | | | | 2.66 | 41,400 | 297,000 | | | | | |
| 734.8 | | | 45,100 | 1330 | 34.0 | | 0.025 | 6.24 | 281,500 | | | | 6.84 | 308,600 | 14,440,000 | | | | | |
| | | | | | | | | | 318,300 | 0.000120 | 0.000099 | 0.24 | | 350,000 | 14,737,000 | 42.1 | 0.65 | - 0.30 | - 0.06 | 734.76 |
| 10 | 365.34 | | 22,600 | 1200 | 18.8 | | 0.080 | 2.11 | 48,600 | | | | 2.06 | 46,700 | 196,000 | | | | | |
| 736.6 | | | 45,300 | 1310 | 34.6 | | 0.025 | 6.31 | 285,600 | | | | 6.13 | 277,800 | 10,440,000 | | | | | |
| | | | 9,600 | 519 | 18.0 | | 0.040 | 2.65 | 26,200 | | | | 2.56 | 25,500 | 170,000 | | | | | |
| | | | | | | | | | 360,000 | 0.00084 | | | | 350,000 | 10,806,000 | 30.9 | 0.48 | | | 736.77 |
| | Fairfax Bridge - 20' (Use Section No. 10) | | | | | | | | | | | | | | | | | | | |
| | | 2360' | | | | | | | | | | | | | | | | | | |
| 736.1 | | | 23,800 | 1200 | 20.8 | | 0.080 | 2.16 | 50,700 | | | | 2.02 | 47,700 | 197,000 | | | | | |
| | | | 46,100 | 1310 | 36.2 | | 0.025 | 6.26 | 283,800 | | | | 6.00 | 276,400 | 9,950,000 | | | | | |
| | | | 10,300 | 521 | 19.6 | | 0.040 | 2.70 | 27,800 | | | | 2.54 | 25,900 | 167,000 | | | | | |
| | | | | | | | | | 372,100 | 0.000086 | 0.000091 | 0.22 | | 350,000 | 10,314,000 | 29.5 | 0.46 | 0.02 | 0.24 | 736.01 |
| | Fairfax Bridge | | | | | | | | | | | | | | | | | | | |
| | | 20' | | | | | | | | | | | | | | | | | | |
| | 364.0 | | 9,010 | 730 | 12.6 | | 0.030 | 1.70 | 16,800 | | | | 1.99 | 19,800 | 78,000 | | | | | |
| 736.0 | | | 48,800 | 1640 | 29.7 | | 0.025 | 5.70 | 278,300 | | | | 6.67 | 325,400 | 14,480,000 | | | | | |
| | | | 2,340 | 168 | 14.2 | | 0.030 | 1.78 | 4,200 | | | | 2.06 | 4,800 | 20,000 | | | | | |
| | | | | | | | | | 299,300 | 0.000137 | 0.000113 | 0.01 | | 350,000 | 14,578,000 | 41.7 | 0.65 | - 0.18 | - 0.16 | 736.62 |
| | Fairfax Bridge ≠ 20' (Use Section No. 10) | | | | | | | | | | | | | | | | | | | |
| | | 20' | | | | | | | | | | | | | | | | | | |
| 736.2 | | | 23,800 | 1200 | 18.8 | | 0.080 | 2.16 | 50,700 | | | | 2.02 | 47,700 | 197,000 | | | | | |
| | | | 46,300 | 1310 | 35.2 | | 0.025 | 6.30 | 293,800 | | | | 6.00 | 276,400 | 9,950,000 | | | | | |
| | | | 10,200 | 521 | 19.6 | | 0.040 | 2.70 | 27,800 | | | | 2.54 | 25,900 | 167,000 | | | | | |
| | | | | | | | | | 372,100 | 0.000086 | 0.000113 | 0.01 | | 350,000 | 10,314,000 | 29.5 | 0.46 | 0.18 | 0.20 | 736.08 |
| | | 2240' | | | | | | | | | | | | | | | | | | |
| 11 | 365.87 | | 22,100 | 2180 | 14.7 | | 0.080 | 1.78 | 37,400 | | | | 2.60 | 63,800 | 364,000 | | | | | |
| 740.0 | | | 29,200 | 852 | 34.3 | | 0.025 | 6.28 | 303,300 | | | | 9.13 | 266,300 | 22,300,000 | | | | | |
| | | | | | | | | | 340,700 | 0.000211 | 0.000130 | 1.24 | | 350,000 | 22,764,000 | 68.0 | 1.01 | - 0.25 | 0.66 | 736.72 |

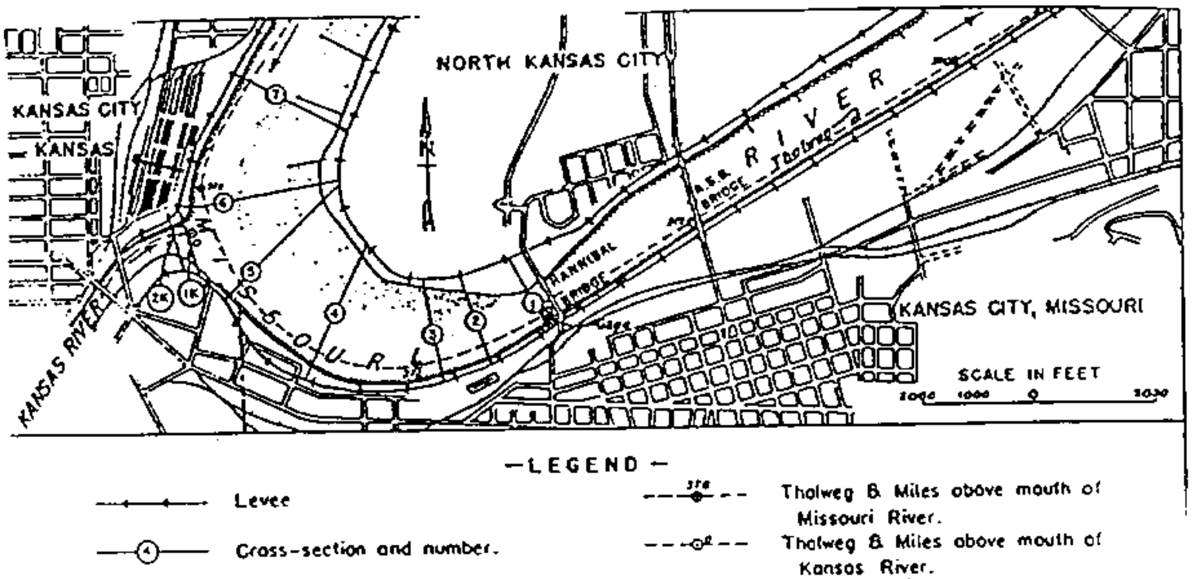


**Figure 6-10. Index map, Missouri and Kansas Rivers at Kansas City, Missouri**

**Table 6-3**
**Tabulation of Hydraulic Elements, Missouri and Kansas Rivers at Kansas City**

| Section No. | River Mile | W.S. Elev. | A | P | R | n | $K' \times 10^{-6}$ | $\frac{(K')^3}{(\Sigma K')^2}$ | $A^{-2} \times 10^{10}$ | $(9) \times (10) \times 10^{10}$ | $F \times 10^{12}$ | $K \times 10^{15}$ | Lw | $F' \times 10^{12}$ | $L_1$ | $F'' \times 10^{12}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 |
| 1 | 377.58 | 752 | 6,000 | 496 | 12.1 | .05 | .9 | .0000325 | 278 | .01 | | | 1060 | | | |
| | | | 38,100 | 910 | 41.9 | .025 | 27.3 | .907 | 6.89 | 6.25 | | | | | | |
| | | | | | | | 28.2 | | | 6.26 | 9.74 | 1.26 | | 10.41 | | |
| | | 753 | 6,500 | 499 | 13.0 | .05 | 1.1 | .000052 | 237 | .01 | | | | | | |
| | | | 39,000 | 911 | 42.8 | .025 | 28.3 | .892 | 6.57 | 5.86 | | | | | | |
| | | | | | | | 29.4 | | | 5.87 | 9.13 | 1.16 | | 8.74 | | |
| | | 754 | 7,300 | 502 | 13.9 | .05 | 1.2 | .000060 | 204 | .01 | | | | | | |
| | | | 39,900 | 912 | 43.7 | .025 | 29.4 | .885 | 6.28 | 5.56 | | | | | | |
| | | | | | | | 30.6 | | | 5.57 | 8.67 | 1.07 | | 9.24 | | |
| 2 | 377.78 | 752 | 2,500 | 208 | 12.0 | .05 | .4 | .0000044 | 1800 | .01 | | | 845 | | 1060 | |
| | | | 41,000 | 1,320 | 31.0 | .025 | 24.0 | .951 | 5.95 | 5.66 | | | | | | |
| | | | | | | | 24.4 | | | 5.67 | 8.82 | 1.48 | | 9.53 | | 7.93 |
| | | 753 | 2,700 | 213 | 12.4 | .05 | .4 | .0000038 | 1370 | .01 | | | | | | |
| | | | 42,300 | 1,325 | 32.0 | .025 | 25.3 | .954 | 5.59 | 5.33 | | | | | | |
| | | | | | | | 25.7 | | | 5.34 | 8.31 | 1.50 | | 8.94 | | 7.51 |
| | | 754 | 2,900 | 217 | 13.4 | .05 | .5 | .0000063 | 1190 | .01 | | | | | | |
| | | | 43,600 | 1,330 | 32.8 | .025 | 26.6 | .946 | 5.26 | 4.98 | | | | | | |
| | | | | | | | 27.1 | | | 4.99 | 7.76 | 1.36 | | 8.33 | | 7.04 |
| 3 | 377.94 | 752 | 1,200 | 120 | 10.0 | .05 | .2 | .0000004 | 6940 | .00 | | | 2000 | | 845 | |
| | | | 48,300 | 1,577 | 30.6 | .025 | 28.0 | .979 | 4.29 | 4.20 | | | | | | |
| | | | | | | | 28.2 | | | 4.20 | 6.53 | 1.26 | | 7.83 | | 6.00 |
| | | 753 | 1,300 | 124 | 10.4 | .05 | .2 | .0000003 | 5920 | .00 | | | | | | |
| | | | 49,900 | 1,581 | 31.5 | .025 | 29.6 | .988 | 4.02 | 3.94 | | | | | | |
| | | | | | | | 29.8 | | | 3.94 | 6.13 | 1.12 | | 7.28 | | 5.66 |
| | | 754 | 1,400 | 128 | 11.0 | .05 | .2 | .0000003 | 5100 | .00 | | | | | | |
| | | | 51,400 | 1,585 | 32.5 | .025 | 31.0 | .981 | 3.79 | 3.72 | | | | | | |
| | | | | | | | 31.2 | | | 3.72 | 5.79 | 1.03 | | 6.85 | | 5.35 |
| 4 | 378.33 | 752 | 61,400 | 2,163 | 28.4 | .025 | 34.7 | 1 | 2.65 | 2.65 | 4.12 | .86 | 1690 | 4.85 | 2060 | 3.23 |
| | | 753 | 63,500 | 2,172 | 29.2 | .025 | 35.8 | 1 | 2.48 | 2.48 | 3.86 | .78 | | 4.52 | | 3.06 |
| | | 754 | 65,700 | 2,181 | 30.1 | .025 | 37.8 | 1 | 2.32 | 2.32 | 3.61 | .70 | | 4.20 | | 2.89 |
| 5 | 378.65 | 752 | 7,200 | 745 | 9.8 | .05 | 1.0 | .000026 | 193 | .01 | | | | | 1690 | |
| | | | 59,100 | 2,084 | 28.5 | .025 | 32.9 | .914 | 2.88 | 2.61 | | | | | | |
| | | | | | | | 33.9 | | | 2.62 | 4.08 | .88 | | | | 3.34 |
| | | 753 | 7,800 | 749 | 10.6 | .05 | 1.1 | .000029 | 160 | .00 | | | | | | |
| | | | 61,200 | 2,085 | 29.3 | .025 | 34.6 | .910 | 2.67 | 2.43 | | | | | | |
| | | | | | | | 35.7 | | | 2.43 | 3.78 | .78 | | | | 3.12 |
| | | 754 | 8,700 | 753 | 11.6 | .05 | 1.2 | .000032 | 132 | .00 | | | | | | |
| | | | 63,300 | 2,087 | 30.3 | .025 | 36.6 | .908 | 2.50 | 2.27 | | | | | | |
| | | | | | | | 37.8 | | | 2.27 | 3.53 | .70 | | | | 2.94 |

*b. Mixed flow regimes.* It is unusual to find a reach where the flow is consistently supercritical. Constrictions and local reductions in cross-sectional area in a stream having an overall slope approaching critical slope can cause the flow regime to oscillate back and forth from supercritical to subcritical. Molinas and Trent (1991) have developed a backwater model which locates changes in flow regime and performs the water surface profile calculations once the regime transition points have been identified.

## 6-19. Ice-covered Streams

*a. Ice stability.* Ice stability analysis by Canadian and American researchers has shown that ice covers and the formation of ice jams are a complex process that is a function of relative stream dimensions, ice properties, and the velocity of flow. Various researchers have categorized ice-covered streams as narrow, wide, deep, and shallow in accordance with criteria that includes velocity, width, depth, and ice thickness.

(1) Pariset et al. (1966) present an ice stability criterion which is suitable for analysis of cohesionless-ice-covered wide rivers. Spring breakup ice is considered to possess negligible cohesion, and is approximately analyzed by Pariset's criterion. Calkins (1978) indicates that Pariset's Equations are appropriate for deep streams. He suggests that, as a rule of thumb, a river can be considered to be deep if the depth of flow is greater than 12 feet.

(2) Pariset's 1966 paper presents the following dimensionless stability criteria "$X$" for analyzing the ratio of the thickness "$h$" of ice to the upstream open water depth "$H$." (This is shown graphically in Figure 6-11.)

$$X = \frac{Q^2}{C^2 B H^4} \qquad (6\text{-}4)$$

where

$X$ = ice stability indicator
$Q$ = discharge
$C$ = Chezy coefficient
$B$ = stream width
$H$ = upstream depth

*b. Ice-covered streams.* Ice cover occurring on a small stream may have sufficient strength to completely bridge the stream during low flow, creating an approximate closed conduit condition. During high flows ice may be held in place by rocks or trees, and as flow rises, open channel conditions may occur above the ice, and pressure flow may occur beneath the ice. Ice covers wide stream floats, and is free to rise and fall with changing discharge.

(1) Profiles may be computed for ice-covered streams by normal standard-step backwater calculations if allowance is made for the flow area blocked by the ice, and if the increased wetted perimeter is accounted for. Hydraulic roughness values must also be adjusted to account for differences in roughness between the ice and the stream bed. The position of the floating ice relative to the free water surface (piezometric head) is determined by the specific gravity of the ice; a typical value is approximately 0.92. Figure 6-12 shows pertinent hydraulic parameters of an ice-covered stream.

$A$ = open flow area under the ice
$P_b$ = wetted perimeter of the channel
$B$ = wetted perimeter of the ice cover
$n_b$ = Manning's $n$ value for the stream bed
$n_i$ = Manning's $n$ value for the ice cover
$R$ = hydraulic radius

$$R = \frac{A}{P_b} \qquad \text{(open channel)} \quad (6\text{-}5)$$

$$R = \frac{A}{P_b + B} \qquad \text{(ice-covered channel)} \quad (6\text{-}6)$$

(2) For wide ice-covered channels, the total wetted perimeter ($W_p + B$) is double the wetted perimeter for the same flow area of an open channel. Thus, the resulting hydraulic radius is half that for an open channel. The increased wetted perimeter is the principal reason that an ice-covered stream requires a greater depth topass an equivalent discharge when compared to a stream flowing under open channel conditions.

$$n_c = \frac{(n_i^{3/2} + n_b^{3/2})^{2/3}}{2} \qquad (6\text{-}7)$$

where

$n_c$ = composite Manning's $n$ value
$n_b$ = stream bed Manning's $n$ value
$n_i$ = ice Manning's value

*c. Ice jams.* A number of researchers have classified ice jams with the different classification schemes depending on the season, ice type, and river width. The

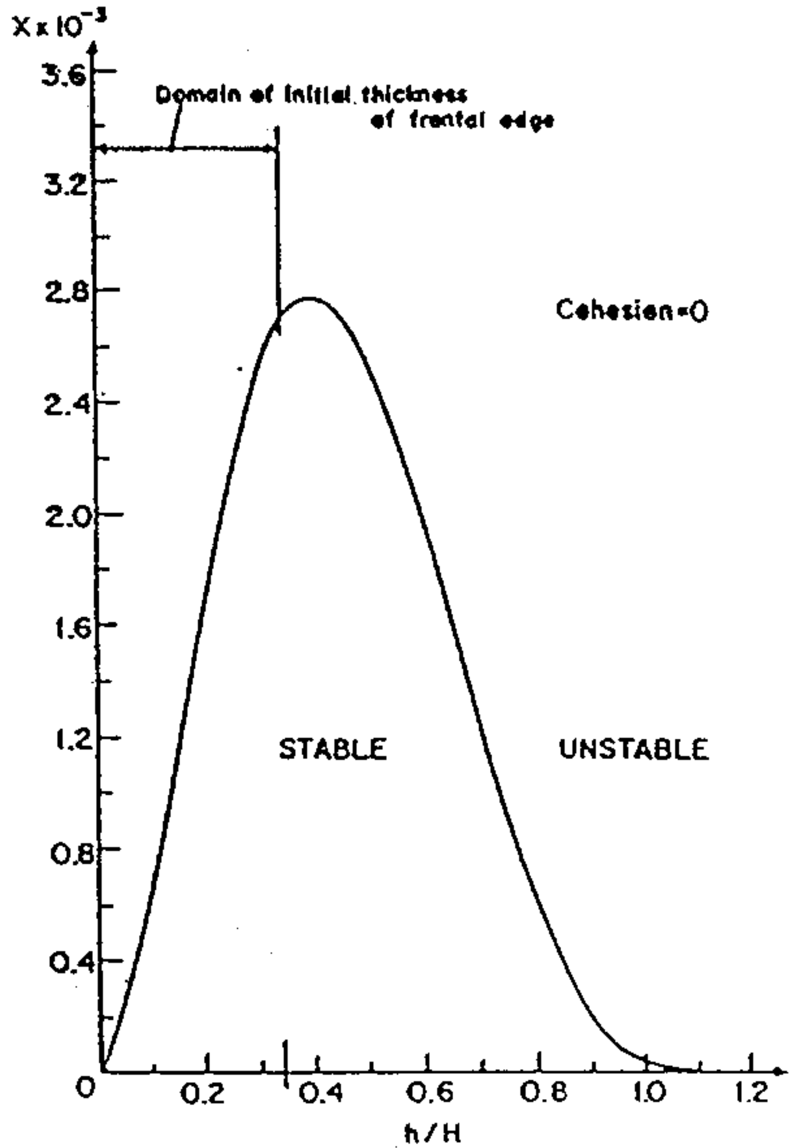


**Figure 6-11. Stability function of ice cover for deep, wide channels**

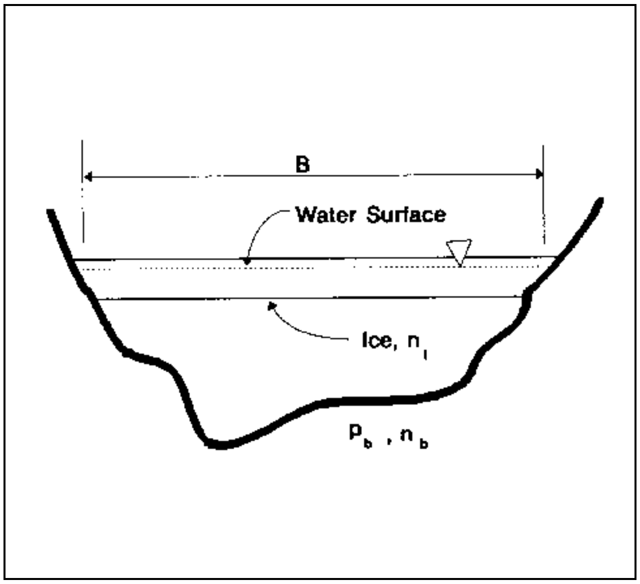


**Figure 6-12. Hydraulic parameters of an ice-covered stream**

primary objective of an ice jam analysis is to predict location, expected thickness and length, associated water levels, and duration.

(1) The locations of ice jams have been identified in the past by prior occurrences at a particular site. Out of a listing of 20 ice jam sites in Vermont, the one common feature that stands out at 14 of the sites is the presence of a relatively long backwater condition. At five sites, two or more streams form junctions; three of these sites are also at the end of a backwater section. Two sites have no structures influencing the jams, but have an almost annual occurrence. At one site, jams form at an obvious channel enlargement, and at the other jams form at an exposed ledge that crops out just upstream of a island. Two ice jam sites have no noticeable physical irregularities in the stream channel geometries, but appear to have relatively mild slopes.

(2) The length and thickness of an ice jam is governed by many factors. One study of ice jam lengths and volumes for streams in the northeastern U.S. showed that the ice jam length did not exceed 10 percent of the upstream river length which contributed ice to the jam.

(3) An estimate for volume of ice in an ice jam can be expressed as

$$V = (1 - C_i)L_r h \tag{6-8}$$

where

$V$ = ice volume in the jam
$C_i$ = coefficient of ice loss
$L_r$ = length of river contributing ice
$h$ = ice cover thickness at breakup

The ice loss coefficient has been computed for some streams in northern New England as ranging from 0.95 to 0.1. The high ice loss coefficient of 0.95 reflected a long river reach with many tributaries and a significant loss of ice to the river banks. The lower ice loss coefficient is for an ice jam in a short river length. Each ice jam site will have a different ice loss coefficient that will be consistent from year to year.

(4) Figure 6-13 shows the average jam depth $h_j$ as a function of position within the normalized jam length $L_r$ for two jams on narrow, steep rivers. The ice jam depth is expressed in multiples of the ice cover thickness prior to breakup, i.e., $h_j/h$. If the initial ice cover is 2 feet, then the ice thickness at the toe of the jam would be roughly 8 feet.

(5) The length of the ice jam $L_j$ can be computed if no records are available by making an assumption about the ice thickness distribution and the volume of ice reaching the site. Using a very simple ice jam length thickness distribution as constant over the length of the jam of $h_j = 2h$, the ice jam length can be computed by dividing the expected volume of ice by the thickness distribution function, yielding

$$L_j = \frac{(1 - C_i)B}{2} \tag{6-9}$$

(6) Figure 6-14 shows the type of variation one can expect in ice jam thickness measurements in one cross section.

(7) The first calculation made in any analysis of an ice jam is to determine the ice volume expected to reach the jam location. The volume can be calculated by measuring river mileage from a USGS topographic map, calculating the expected ice thickness, and determining the average river top width. Once a volume has been calculated, engineering judgment must be used to determine the actual amount of ice reaching the site. A good first approximation is 10 percent.

| Location | $L_j$ | | $h_j$ | |
|---|---|---|---|---|
| | ft | m | ft | m |
| (O)Tunbridge | 3000 | 915 | 1.5 | 0.46 |
| (•)Woodstock | 9200 | 2804 | 1.7 | 0.52 |

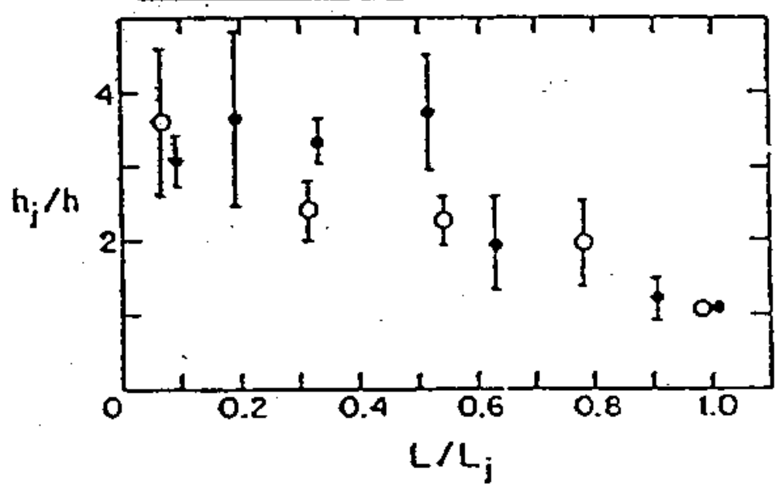


**Figure 6-13. Nondimensional ice jam thickness versus its relative length (narrow, steep rivers)**

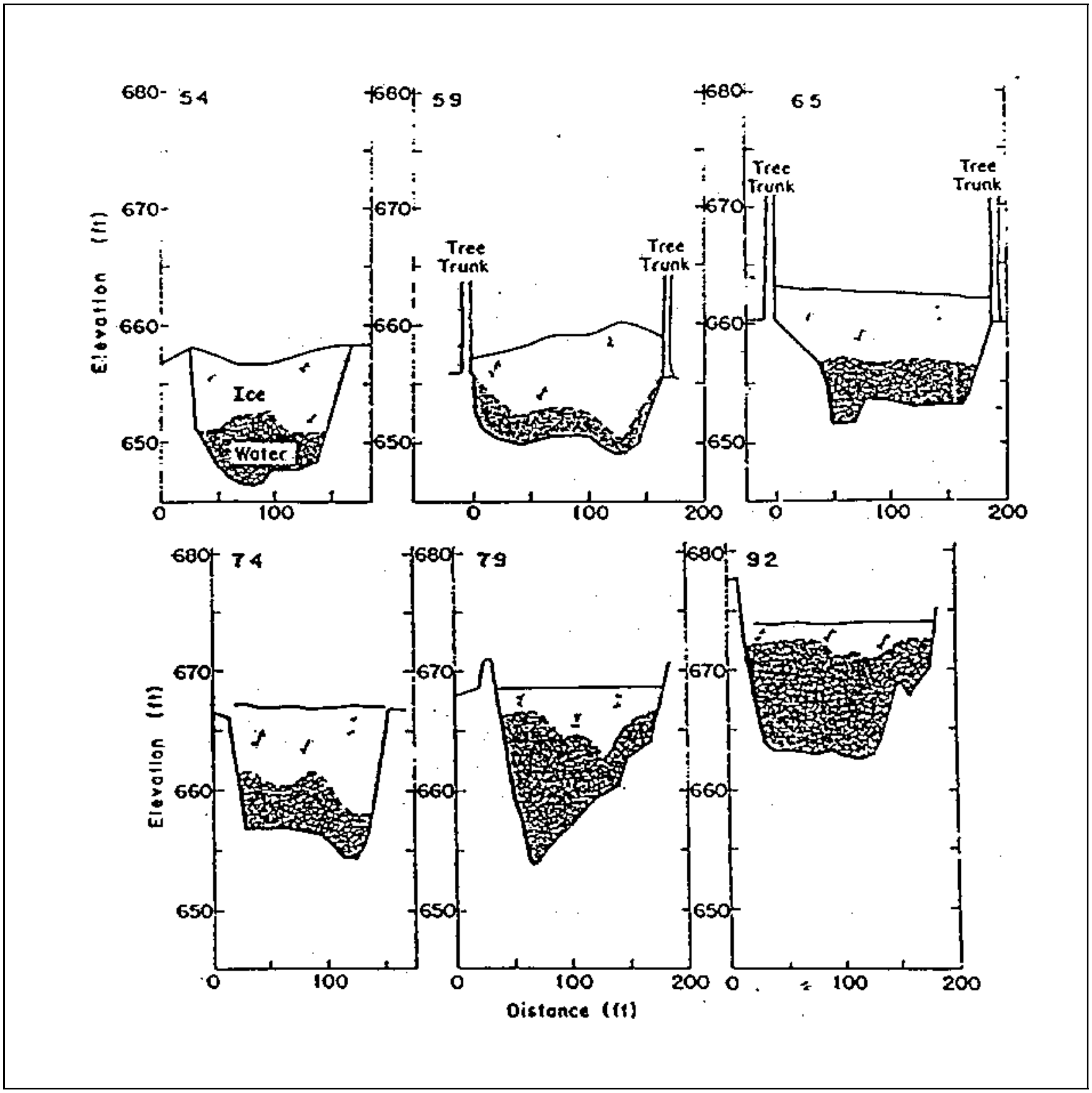


Figure 6-14. Typical ice jam sections on a shallow stream

# Chapter 7
# Water Surface Profiles With Movable Boundaries

*Section I*
*Introduction*

## 7-1. Similarities and Differences Between Fixed and Mobile Bed Computations

The computation of water-surface profiles for flow over a movable boundary differs from fixed bed water-surface profile computations as illustrated in Figures 7-l and 7-2. In both cases a study reach is identified and boundaries are drawn around it to form model limits. Within those model limits, the geometry and loss coefficients are assembled to make a digital model of the study area. A physical analogy at this point is an empty flume.

*a. The fixed-bed solution.* As can be seen from the basic equations governing steady gradually varied flow over a fixed bed (see Chapter 6), the solution requires that two values be given, usually water discharge and water surface elevation. In mathematical terminology, the flow entering the model and the tailwater elevation are called "boundary conditions." A physical analogy is opening a valve to let water enter a flume and regulating the tailgate so that flow leaves the flume at the desired depth. The boxes in Figure 7-1 depict the solution process by showing the typical hydraulic parameters, water velocity, depth, width and slope, with arrows indicating the sequence of the computations.

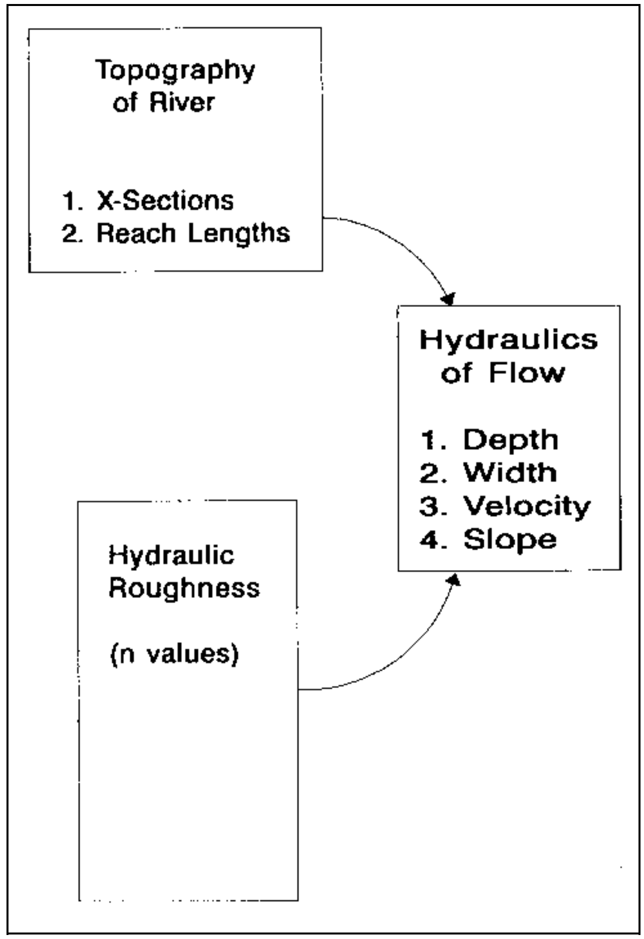


**Figure 7-1. Fixed bed model**

*b. The mobile-bed solution.* The addition of a mobile bed increases the number of processes which must be included in a numerical model. Sediment transport, bed roughness, bed armor, bed surface thickness, bed material sorting, bed porosity, and bed compaction equations are required in addition to the sediment continuity equation which defines the sediment exchange rate between the water column and bed surface. The number of additional equations causes a major increase in complexity. That is not the most significant difference between fixed and mobile bed numerical computations, however. The most important difference is that the cross section shape and bed *n* value become functions of the flow hydraulics. Consequently, a feedback loop is created as illustrated by the arrows in Figure 7-2. The uncertainty about *n* values substantially complicates numerical modeling of mobile boundary problems. There are other major gaps. For example, the bed sorting process which occurs when a mixture of sediment sizes is transported is poorly understood. Also, because sediment is transported primarily in the channel, mobile bed computations must maintain an accurate distribution of flow between the left overbank, channel, and right overbank at each cross section, as well as a history of how the flow arrived at that location in the cross section. It is only necessary to balance energy in a fixed bed computation to solve for the water surface elevation.

*Section II*
*Theoretical Basis*

## 7-2. Sediment Transport Functions

Before 1942 much of the work in sediment transport was influenced by DuBoys (1879). He proposed the idea of a

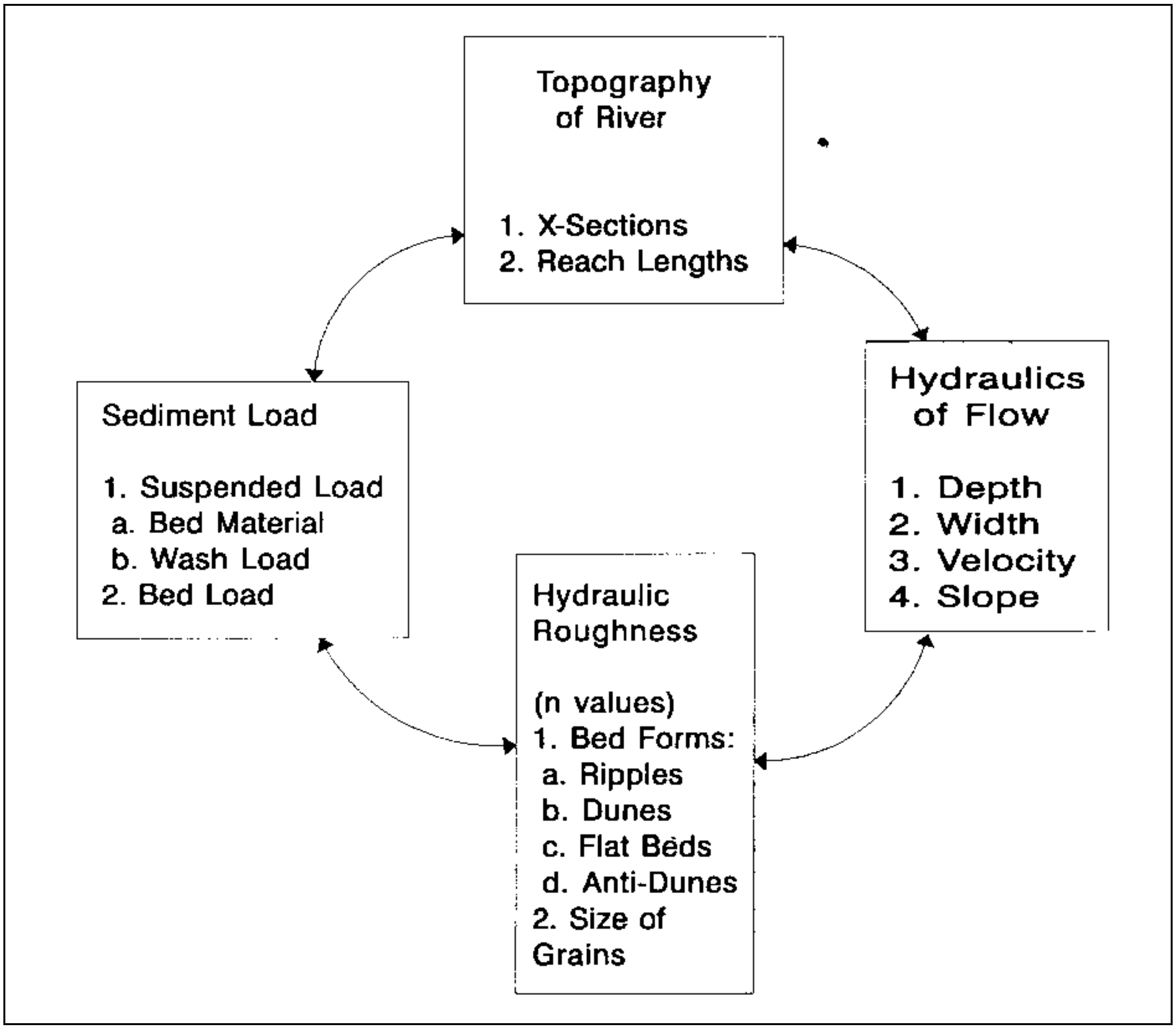


**Figure 7-2. Movable bed model**

bed shear stress and visualized a process by which the bed material moved in layers. The depth of movement was that required for the velocity to become zero. The DuBoys formula for sediment transport is described in ASCE (1975). A major change in the approach to predicting sediment transport was proposed by Einstein (1942) when he presented a transport formula based on probability concepts in which the grains were assumed to move in steps with the average step length proportional to the sediment grain size. The Einstein Bed-Load Function, Einstein (1950) embodies those concepts.

*a. Einstein's concepts.* Einstein described bed material transport as follows:

> *The least complicated case of bed-load movement occurs when a bed consists only of uniform sediment. Here, the transport is fully defined by a rate. Whenever the bed consists of a mixture the transport must be given by a rate and a mechanical analysis or by an entire curve of transport against sediment size. For many years this fact was neglected and the assumption was made that the mechanical analysis of transport is identical with that of the bed. This assumption was based on observation of cases where the entire bed mixture moved as a unit. With a larger range of grain diameters in the bed, however, and especially when part of the material composing the bed is of a size that goes into suspension, this assumption becomes untenable. Some examples of this type of transport are given in the flume experiments described on pp. 42-44 of this publication.*

*The mechanical analysis of the material in transport is basically different from that of the bed in these experiments. This variation of the mechanical analysis will be described by simply expressing in mathematical form the fact that the motion of a bed particle depends only on the flow and its own ability to move, and not on the motion of any other particles.* (Einstein 1950, p. 32)

(1) Einstein's hypothesis that motion of a bed particle depends only on the flow and its own ability to move and not on the motion of any other particles allowed him to describe the equilibrium condition for bed material transport mathematically as two independent processes: deposition and erosion. He proposed an "equilibrium condition," and defined it as the condition existing when "For each unit of time and bed area the same number of a given type and size of particles [are] deposited in the bed as are scoured from it" (Einstein 1950, p. 32).

(2) Although Einstein's work is classic and presents a complete view of the processes of equilibrium sediment transportation, it has been more useful for understanding those processes than for application, partially because of the numerical complexity of the computations. Many other researchers have contributed sediment transport functions - always attempting to develop one which is reliable when compared with a variety of field data. The resulting functions are numerous, yet no single function has proved superior to the others for all conditions. Therefore, the following functional form is presented here to show the importance of various parameters.

$$G = f\,(U,\ d,\ S_e\ ,\ B,\ D_{eff}\ ,\ SG_s\ ,\ G_{sf}\ ,\ D_{si}\ ,\ P_i\ ,\ SG_f\ ,\ T\ ) \qquad \text{(Sediment Transport)} \quad (7\text{-}1)$$

where

$B$ = effective width of flow
$d$ = effective depth of flow
$D_{eff}$ = effective particle diameter of the mixture
$D_{si}$ = geometric mean of particle diameters in each size class $i$
$G$ = total bed material discharge rate in units of weight/time (e.g. tons/day)
$G_{sf}$ = grain shape factor
$P_i$ = fraction of particles of the $i^{th}$ size class that are found in the bed
$S_e$ = slope of energy grade line
$SG_f$ = specific gravity of fluid
$SG_s$ = specific gravity of sediment particles
$T$ = water temperature
$U$ = flow velocity

Of particular interest are the groups of terms: hydraulic parameters ($U$, $d$, $S_e$ , $B$), sediment particle parameters ($D_{eff}$ , $SG_s$ , $G_{sf}$ ), sediment mixture parameters ($D_{si}$ , $P_i$ ), and fluid properties ($SG_f$ , $T$).

*b. Selection of a sediment transport function.* As mentioned above, numerous transport functions have been developed with the aim of computing the rate and size distribution of the transport of bed material, given the hydraulics and bed material gradation (ASCE 1975). As it cannot be stated which one is the "best" to use given a particular situation, the engineer should become familiar with how the functions were derived, what types of data they have been compared to (laboratory flume versus river measurements), and past usage. A recent study (Yang and Wan 1991) rated the accuracy of several transport functions compared with both laboratory and river data and concluded that, for river data, the accuracy in descending order was Yang, Toffaleti, Einstein, Ackers and White, Colby, Laursen, Engelund, and Hansen. It also states that the rating does not guarantee that any particular formula is superior to others under all flow and sediment conditions. Another study (Gomez and Church 1989) favored the formulas of Einstein, Parker, and Ackers-White for gravel bed rivers. An "applicability index" based on river characteristics was developed by Williams and Julien (1989). The WES-SAM (USAEWES 1991) package offers a procedure to aid in the selection. It is based on screening of the various transport functions using information from past studies that compared computed and calculated transport rates and the hydraulic characteristics of the particular stream. Use of such an approach is documented by U.S. Army Corps of Engineers (1990e). The engineer should be aware that different transport functions will probably yield different answers. The impact will most likely be greater on transport rates than on computed geometry changes. Extreme situations, such as mud and debris flows, require different analytic techniques, see U.S. Army Corps of Engineers (1990f) for an example.

*c. Numerically modeling the movable boundary problem.* Although sediment discharge formulas appear in a numerical model of the movable boundary problem, there are significant differences between the calculations for sediment discharge and those in a mobile boundary sediment movement model. Table 7-1 summarizes those differences. The words "equilibrium" and "nonequilibrium" condition in this table refer to the exchange of sediment particles between the flow field and the bed. Whereas the bed is the only source of sediment to a sediment transport formula, a sediment movement model

**Table 7-1**
**Sediment Tansport versus a Movable Bed Sedimentation model**

| A. Sediment discharge formulas. | B. Sediment movement models. |
|---|---|
| A1. Require flow intensity, bed roughness, specific gravity of particles, and bed surface gradation. | B1. All of A1 plus inflowing sediment load, geometry over long distances, bedrock locations, and gradations beneath the bed surface. |
| A2. Calculate the equilibrium condition. | B2. All of A2 plus calculate changes in bed profile due to nonequilibrium transport. |
| A3. Functional only for the bed material load. | B3. In the case of sand moving over a gravel bed, models will calculate both the load moving and bed surface gradation required to sustain it. Wash load can be handled in several ways. |

should partition the river into reaches so that both the bed and the inflowing sediment load to the reach are sediment sources to the calculations in that reach. Nonequilibrium conditions are common from one reach to the next because sediment movement tends to be highly variable in both rate and particle size distribution. A mobile bed sedimentation numerical model should calculate transport by size class and keep a continuous accounting of the gradation in the stream bed and on its surface.

(1) To have general applicability a numerical sedimentation model must erode, entrain, transport, deposit and consolidate mixtures of sediment particles for the nonequilibrium case. Einstein did not address the nonequilibrium condition, but his "particle exchange" concept was extended for the HEC-6 numerical sediment movement model as described in Section 7-12.

(2) Sediment movement modeling for most engineering studies does not require tracing the motion of individual particles. Rather, it requires calculating the influence of flow intensity on bed particle behavior, subject to particle size and availability. The objective is to calculate changes in the bed surface elevation in response to nonequilibrium sediment conditions and to feed those changes back into the hydraulic calculation of the flow intensity-sediment load parameters. Some questions dealing with sediment quality cannot be fully addressed, however, without tracing the paths and dispersion of the sediment particles.

*Section III*
*Data Requirements*

## 7-3. General Data Requirements

Two types of data are required. One type records the behavior of the prototype. The other is the data required to operate the numerical model. The first is summarized for completeness. The second, which begins with geometry, is presented in more detail. The project area and study area boundaries should be marked on a project map to delineate the area needing data. Show the lateral limits of the study area and the tributaries. Bed profiles from historical surveys in the project area are extremely valuable for determining the historical trends which the model must reconstitute. Aerial photographs and aerial mosaics of the project area are very useful for identifying historical trends in channel width, meander wave length, rate of bank line movement, and land use in the basin. Gage records contain the annual water delivery to the project area and the water yield from it. They are also useful for establishing the hydraulic parameters of depth, velocity, *n*-value, and the trends in stage-discharge curves in, or close to, the study reach. It is important to work with measured data. Do not regard the "extrapolated" portion of a rating curve as measured data. An example of this is shown in Figure 7-3 where the measured flows are less than 1,850 cfs whereas the project formulation flows range up to 16,000 cfs. Be aware that "measured" data is subject to errors as discussed in sections 5-8

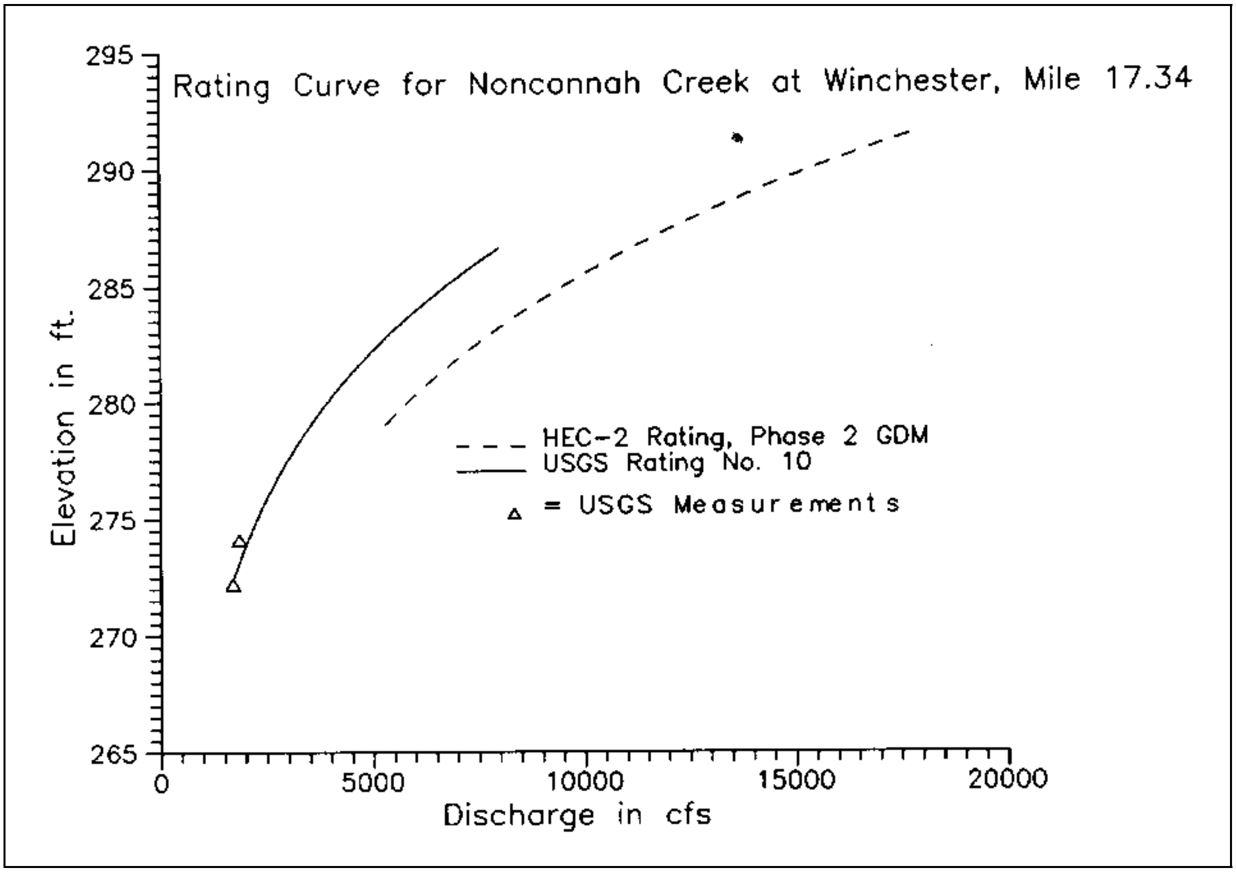


**Figure 7-3. Rating curve at a gage**

and 6-10. Reconnaissance of the project reach is a valuable aid for determining channel morphology, geometric anomalies, the existence of structures, and sediment characteristics of the channel. Include geotechnical and environmental specialists in a field reconnaissance if possible. Document these observations of the prototype in project reports. View as much of the prototype as is feasible and not just at bridge crossings. Hydraulic data such as measured water surface profiles, velocities, and flood limits in the study reach are extremely valuable. Local agencies, newspapers, and residents along the stream are valuable sources of information that can supplement field measurements.

## 7-4. Geometric Data

Mobile bed calculations attempt to determine the water surface and bed surface elevations as they change over time. It is necessary to prescribe the initial geometry. After that, computations aggrade or degrade the cross sections in response to mobile bed theory. The cross sections never change locations.

*a. Cross sections.*

(1) As in fixed bed calculations, it is important to locate the cross sections so they model the channel contractions and expansions. It is particularly important in mobile boundary modeling to also recognize and set conveyance limits. That is, when flow does not expand to the lateral extent of a cross section in the prototype, conveyance limits should be set in the model.

(2) There is no established maximum spacing for cross sections; it depends on both study needs and accuracy requirements related to the particular numerical model being used. Some studies have required distances as short as a fraction of the river width. Others have successfully used sections spaced 10-20 miles apart. The objective is to develop data that will reconstitute the historical response of the streambed profile and capture

key features of the flow and the boundary movement. The usual approach is to start with the same geometry that was developed for fixed bed calculations. Note that, as most fixed bed data sets are prepared to analyze flood flows, they may be biased towards constrictions such as bridges and deficient of reach-typical sections that are important for long term river behavior. There may also be cases when some of these cross sections must be eliminated from the data set to preserve model behavior, such as at deep bends or junctions where the shape is molded by turbulence and not one-dimensional sediment transport, but those are usually exceptions.

*b. River mile.* Show the cross sections on a map, as in Figure 7-4, for future reference. Use of river mile as the cross section identification number is recommended. It is much easier to use or modify old data if the cross sections are referenced by river mile rather than an arbitrary section number.

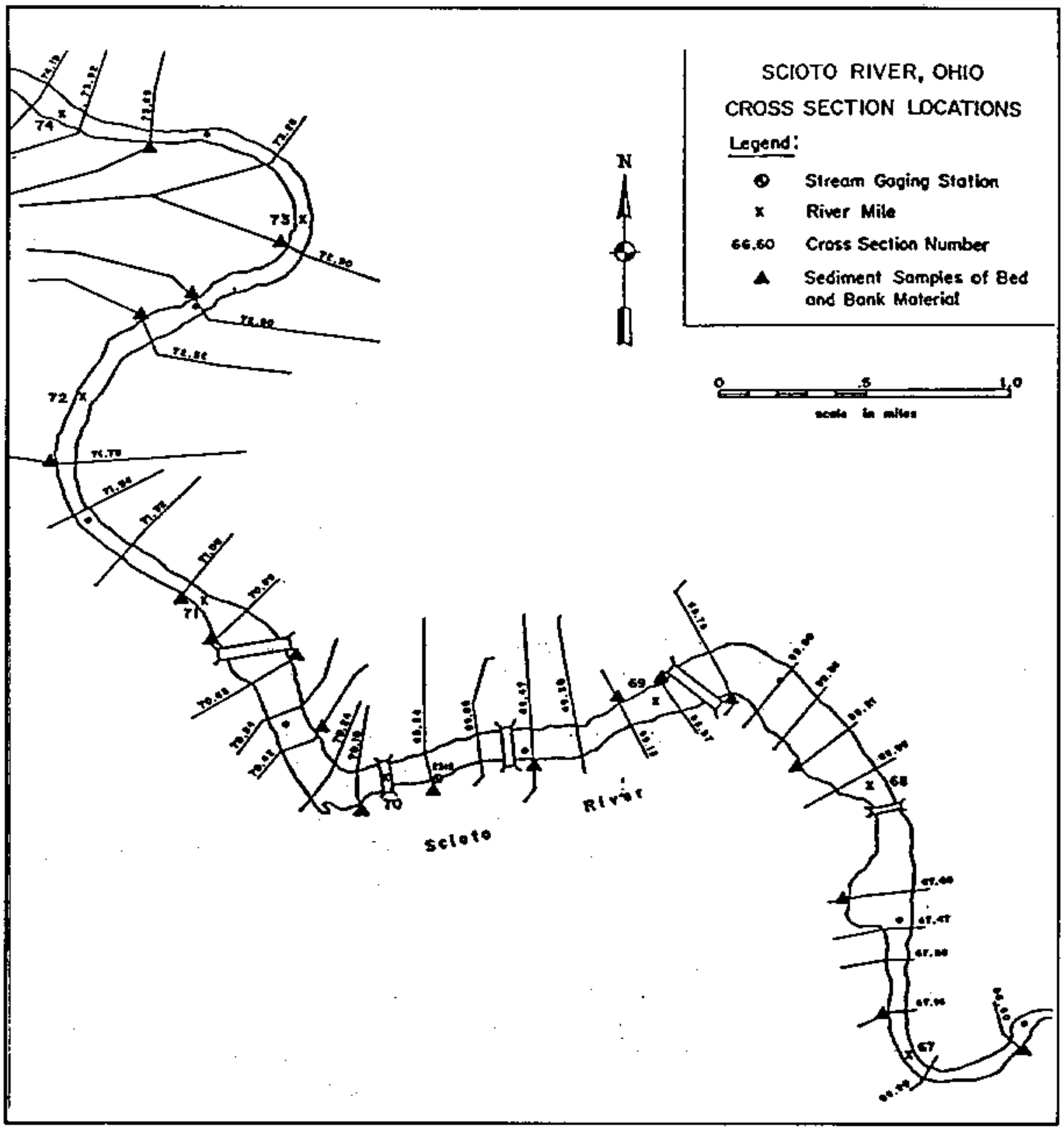


**Figure 7-4. Cross section locations**

## 7-5. Bed Sediment Data

The bed sediment reservoir is the space in the bed of the stream from which sediment can be eroded or on to which it can be deposited. This reservoir occupies the entire width of the channel, and in some cases, the width of the overbank also. It might have a very small depth, however, as in the case of a rock outcrop.

*a. Gradation of the bed sediment reservoir.* It is also necessary to prescribe the gradation of sediment in the bed sediment reservoir.

*b. Conditions data.* The section on "Boundary Conditions Data" (7-6) provides suggestions for selecting sample locations for use in calculating an inflowing sand and gravel discharge rate. This section gives suggestions for selecting locations that also describe development of the armor layer to resist erosion.

(1) For example, in one study two samples were taken in the dry at each of 27 cross sections spaced over a 20 mile reach of the creek. One was from near the water's edge and the other was from the point bar deposits, about half the distance to the bank. These samples were sieved separately and the resulting gradations plotted; see Figures 7-5 and 7-6.

(2) Results from the water's edge samples were used to test for erosion because they were coarser than themid bar samples. The midbar samples were used to test for transport rates.

## 7-6. Boundary Conditions Data

Four types of data are included in this category: inflowing water discharges, inflowing sediment concentrations, inflowing sediment sizes, and elevation of the water surface at the outflow boundary.

*a. Water inflows.* Although an instantaneous water discharge (e.g. a flood peak) may be of interest, it is not sufficient for movable bed analysis because time is a variable in the governing equations and sediment volumes rather than instantaneous rates of movement create channel changes. Consequently, a water discharge hydrograph must be developed. This step can involve manipulations of measured flows, or it can require a calculation of the runoff hydrograph. Historical flows are needed to reconstitute behavior observed in the river, and future flows are needed to forecast the future stream bed profile.

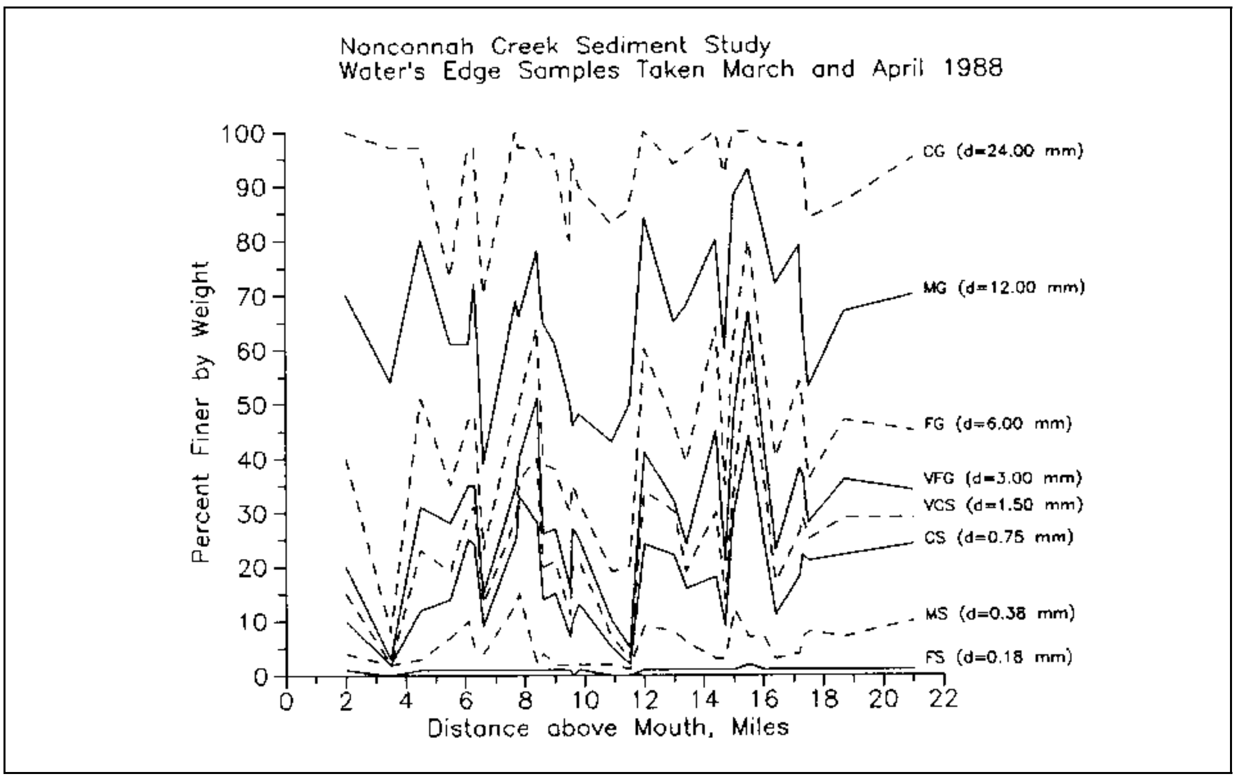


**Figure 7-5. Bed surface gradation based on water edge samples**

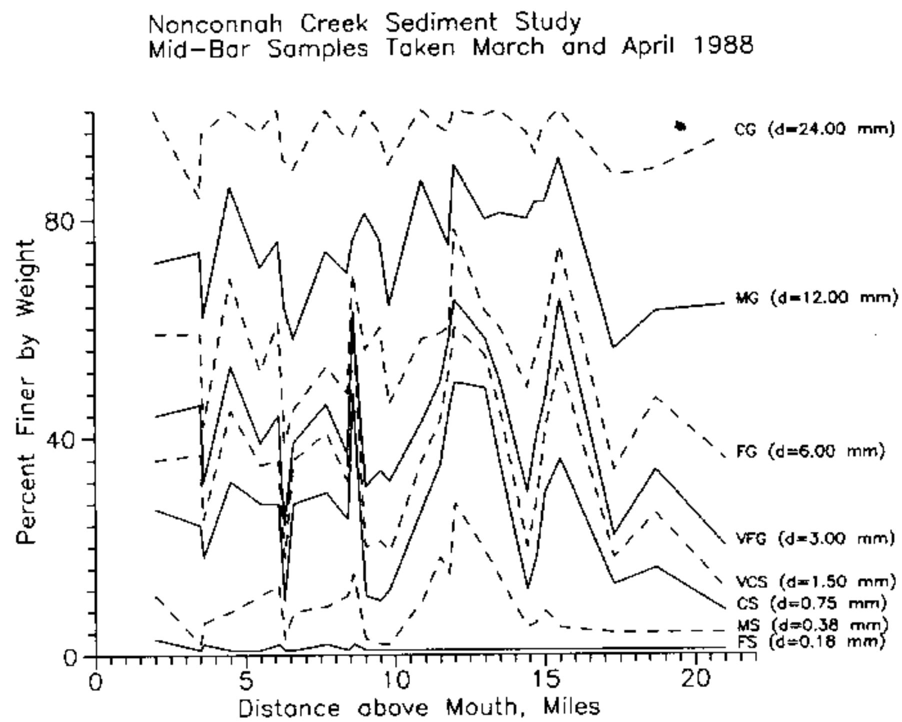


**Figure 7-6. Bed surface gradation based on midbar samples**

(1) The length of the hydrograph period is important. Trends of a tenth of a foot per year of change in bed elevation become significant during a 50- or 100-year project life. A long period hydrograph can become a computational burden. In some cases, data compression techniques may be useful. As an example, Figure 7-7 shows how a year of mean daily flows might be represented by fewer discharges of longer duration.

(2) Tributaries are lateral inflow boundary conditions. They should be located, identified, and grouped as required to define water and sediment distributions. The locations should be shown on the map of the cross section locations. It is important that the water and sediment inflows from all gaged and ungaged areas within the study reach be included. A water balance should be performed for the study period. Keep in mind that a 10 percent increase in water discharge may result in a 20 percent increase in bed material transport capacity. Inflows from ungaged areas must be developed. Drainage area ratios may be used in some cases; in others, however, use or development of a hydrologic model of the basin may be necessary. Document how inflows were determined for those tributaries that were not included in the analysis as individual channels.

*b. Sediment inflows.* The second and third boundary conditions are the inflowing sediment concentration and the fraction of that concentration in each particle size class.

(1) Inflowing sediment concentrations. Occasionally suspended sediment concentration measurements, expressed as milligrams per liter, are available. These are usually plotted against water discharge and often exhibit very little correlation with the discharge; however, use of such graphs is encouraged when developing or extrapolating the inflowing sediment data. As the analysis proceeds, it is desirable in most situations to convert the concentrations to sediment discharge in tons/day and to express that as a function of water discharge as shown in Figure 7-8. A scatter of about 1 log cycle is common in such graphs. The scatter is smaller than on the concentration plot because water discharge is being plotted on both axes. The scatter may result from seasonal effects (e.g., vegetation and fires), random measurement errors, changes in the watershed or hydrology during the measurement period, or other sources. The analyst should carefully examine these data and attempt to understand the shape and variance of the relationship.

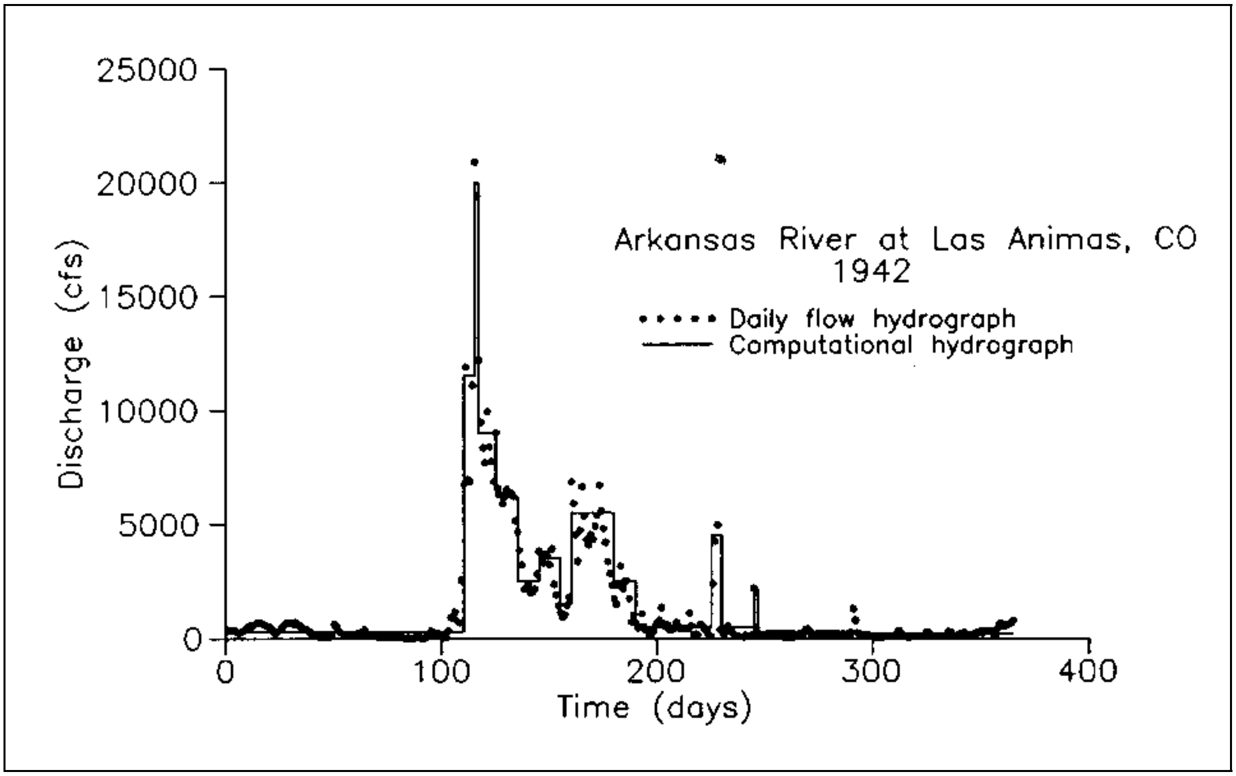


**Figure 7-7. Water discharge histograph**

(2) Grain size classes. The total sediment discharge should be partitioned into size classes for the mobile bed computations. Table 7-2 shows a procedure developed for the Clearwater River at Lewiston, Idaho. Figure 7-9 is the graph of that data set. Note that, due to the availability of various size fractions in the bed and the suspended load gradation for a given flow, the transport rate does not necessarily decrease with increasing particle size. This phenomenon occurs primarily at low flows and may, therefore, be of little consequence to the overall stream behavior.

(3) Calculating sediment inflow with transport theory. When no suspended sediment measurements are available, the inflowing sediment boundary condition must be calculated. That is possible for sand and gravel using mobile bed hydraulics and sediment transport theory. There is no comparable theory for the wash load inflow. When making a calculation for the boundary condition, select the reach of channel very carefully. It should be one approaching the project which has a slope, velocity, width and depth typical of the hydraulics which are transporting the sediment into the project reach. It should also have a bed surface that is in equilibrium with the sand and gravel discharge being transported by the flow. Having located such a reach, sample the bed surface over a distance of several times the channel width. Focus on point bars or alternate bars rather than the thalweg of the cross section. Measure the geometry of that reach. Make the calculation by particle size for the full range of water discharges in the study plan.

(4) Bed material sampling. Figure 7-10 illustrates a typical bed sediment gradation pattern on a point bar. Use such information to determine where to sample to get the bed gradation for a sediment transport calculation. Note that, although the typical grain sizes found on the bar surface form a pattern from coarse to fine, there is no one location which always captures the precise distribution which will represent the entire range of processes in the prototype. The bed gradation governs the calculated sediment discharge. For example, the rate of transport increases exponentially as the grain size decreases (Figure 7-11). There is no simple rule for locating samples. The general rule is "always seek representative samples." That is, very carefully select sampling locations and avoid anomalies which would bias either the calculated sediment discharge or the calculated bed stability against erosion. Samples taken near structures such as bridges

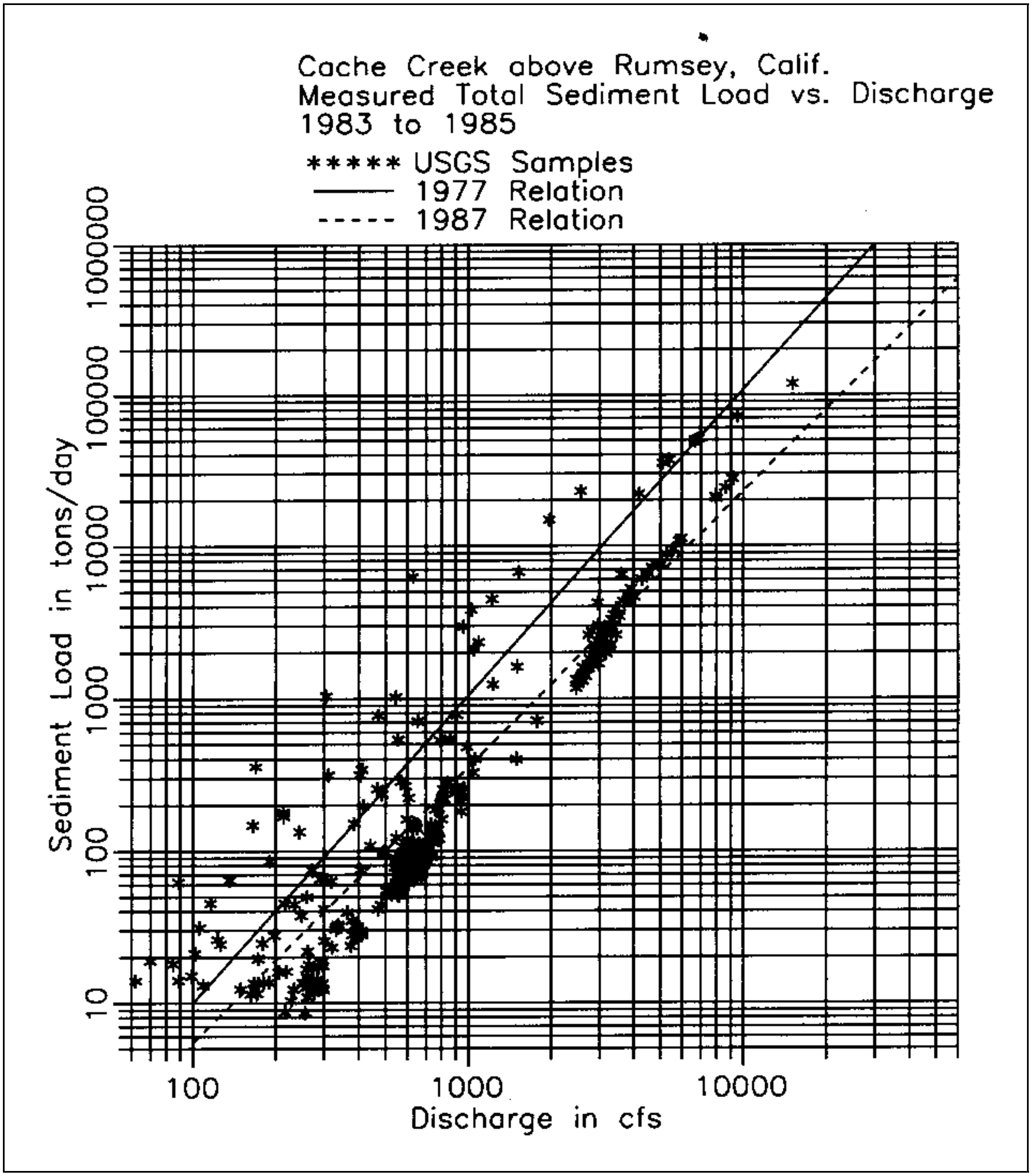


Figure 7-8. Sediment-discharge rating curve

**Table 7-2**
**Distribution of Sediment Load by Grain Size Class**

Water discharge: 35,000 cfs

Total Bed Load, tons/day. . . . . .130
Total Susp. Load, tons/day. . . . . .1,500
Total Sediment Load. . . . . . 1,630

| Grain Size Diameter mm | Classification | Percent Bed Load | Bed Load tons/day | Percent Suspended Load | Suspended Load tons/day | Total Load Column (4) +(6) tons/day |
|---|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) | (7) |
| <.0625 | silt & clay | 0.04 | 0.05 | 54 | 810 | 810 |
| 0.0625-.125 | VFS | 0.10 | 0.13 | 10 | 150 | 150 |
| 0.125-.250 | FS | 2.75 | 4.00 | 13 | 195 | 199 |
| 0.250-.500 | MS | 16.15 | 21.00 | 19 | 285 | 306 |
| 0.500-1 | CS | 13.28 | 17.00 | 4 | 60 | 77 |
| 1-2 | VCS | 1.19 | 2.00 | | | 2 |
| 2-4 | VFG | 1.00 | 1.00 | | | 1 |
| 4-8 | FG | 1.41 | 2.00 | | | 2 |
| 8-16 | MG | 2.34 | 3.00 | | | 3 |
| 16-32 | CG | 6.33 | 8.00 | | | 8 |
| 32-64 | VCG | 23.38 | 30.00 | | | 30 |
| >64 | cobbles & larger | 32.03 | 42.00 | | | 42 |
| TOTAL | | 100.0 | 130.18 | 100.0 | 1,500 | 1,630 |

Notes:

1. The distribution of sizes in the bed load is usually computed using a bed load transport function and field samples of bed material gradation. The bed load rate is rarely measured and may have to be computed.
2. The suspended load and its gradation can be obtained from field measurements.

will rarely be representative of reach transport characteristics.

(5) Sediment inflow from tributaries. The sediment inflow from tributaries is more difficult to establish than it is for the main stem because there is usually less data. The recourse is to assess each tributary during the site reconnaissance. For example, look for a delta at the mouth of the tributary. Look for channel bed scour or deposition along the lower end of the tributary. Look for drop structures or other controls that would aid in stabilizing a tributary. Look for significant deposits if the tributaries have concrete lining. These observations will help guide the development of tributary sediment discharges.

*c. Tailwater elevation.* The final boundary condition specifies the water surface elevation at the downstream end of the study reach. It is referred to as a tailwater elevation because it serves the same purpose as a tailgate on a physical model. It can be a stage-discharge rating curve (Figure 7-3); or it can be a stage hydrograph. The rating curve can be calculated by normal depth if the boundary is in a reach where friction is the control and the water surface slope is approximately constant for the full range of discharges. When a backwater condition exists, such as at the mouth of a tributary or in a reservoir, then use a stage hydrograph as the boundary condition. Be sure it covers the same period of time as the inflow hydrographs.

*d. Boundary condition changes over time.* The above discussion assumes that the inflowing sediment load curves and their particle size distributions, as well as the tailwater rating curve, will not change in the future. That assumption should be justified for each project or appropriate modifications made to the study procedure and numerical model application.

## 7-7. Data Sources

*a. General.* The data that will be needed for the study may come from office files, other federal agencies, state or local agencies, universities, consultants, the team making the field reconnaissance of the project site and

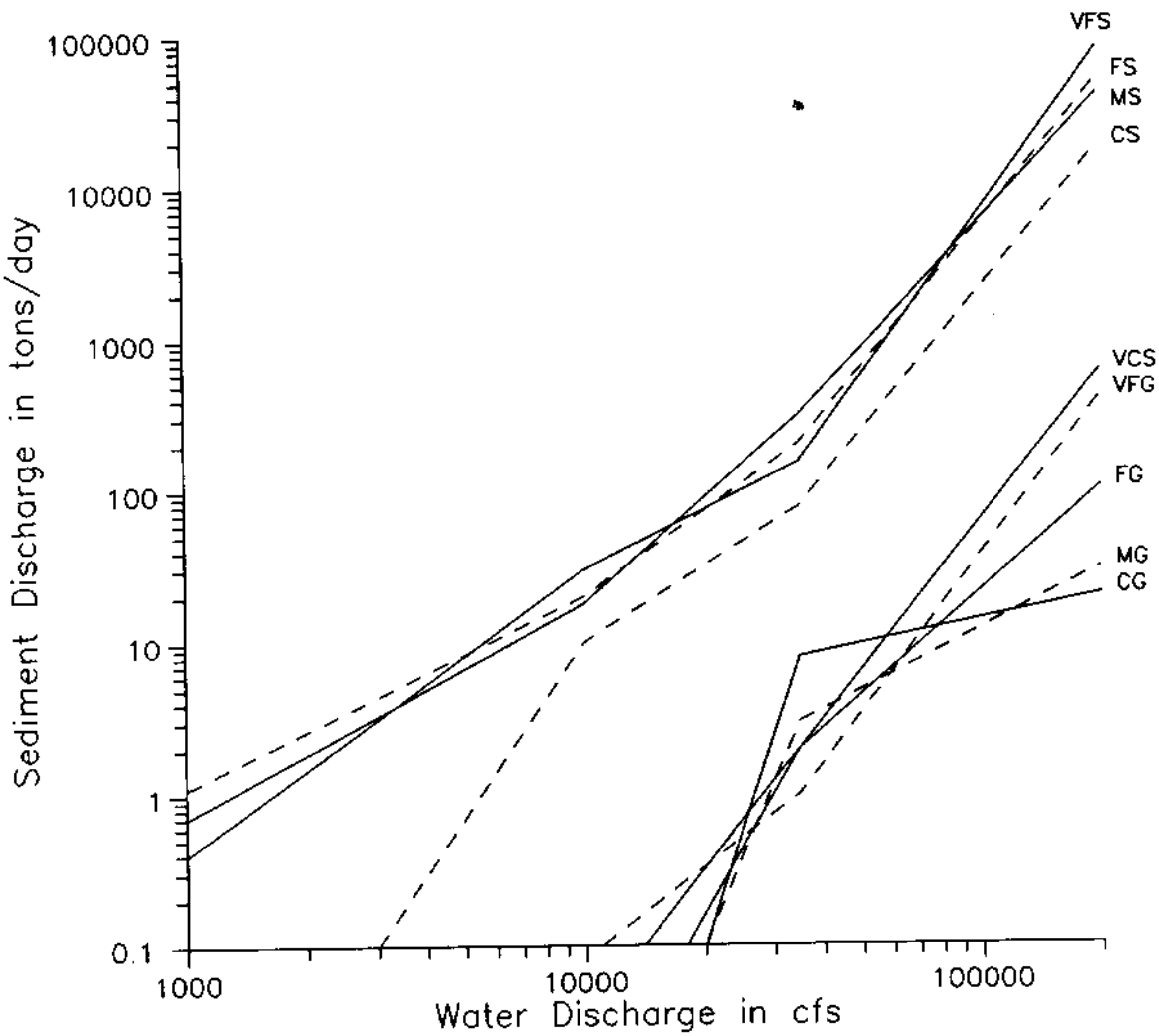


**Figure 7-9. Sediment load curves**

study reach, and surveys initiated specifically for the study.

*b. U. S. Geological Survey (USGS).* USGS topographic maps and mean daily discharges are used routinely in hydraulic and hydrology studies and are also common data sources for sediment studies. Mean daily flows, however, are often not adequate for sediment studies. Data for intervals less than one day or stage-hydrographs for specific events, if needed, can be obtained from strip-chart stage recordings that are available by special request. It may be preferable to use USGS discharge-duration tables rather than developing such in house; these are available from the state office of the USGS. Water quality data sometimes include suspended sediment concentrations and grain size distributions. Published daily maximum and minimum sediment discharges for each year and for the period of record are available as are periodic measurements of particle size gradations for bed sediments.

*c. National Weather Service (NWS).* There are cases where mean daily runoff can be calculated directly from rainfall records and expressed as a flow-duration curve without detailed hydrologic routing. In those cases, use the rainfall data published monthly by the National Weather Service for each state. Hourly and daily rainfall data, depending on the station, are readily accessible. Shorter interval or period-of-record rainfall data can be

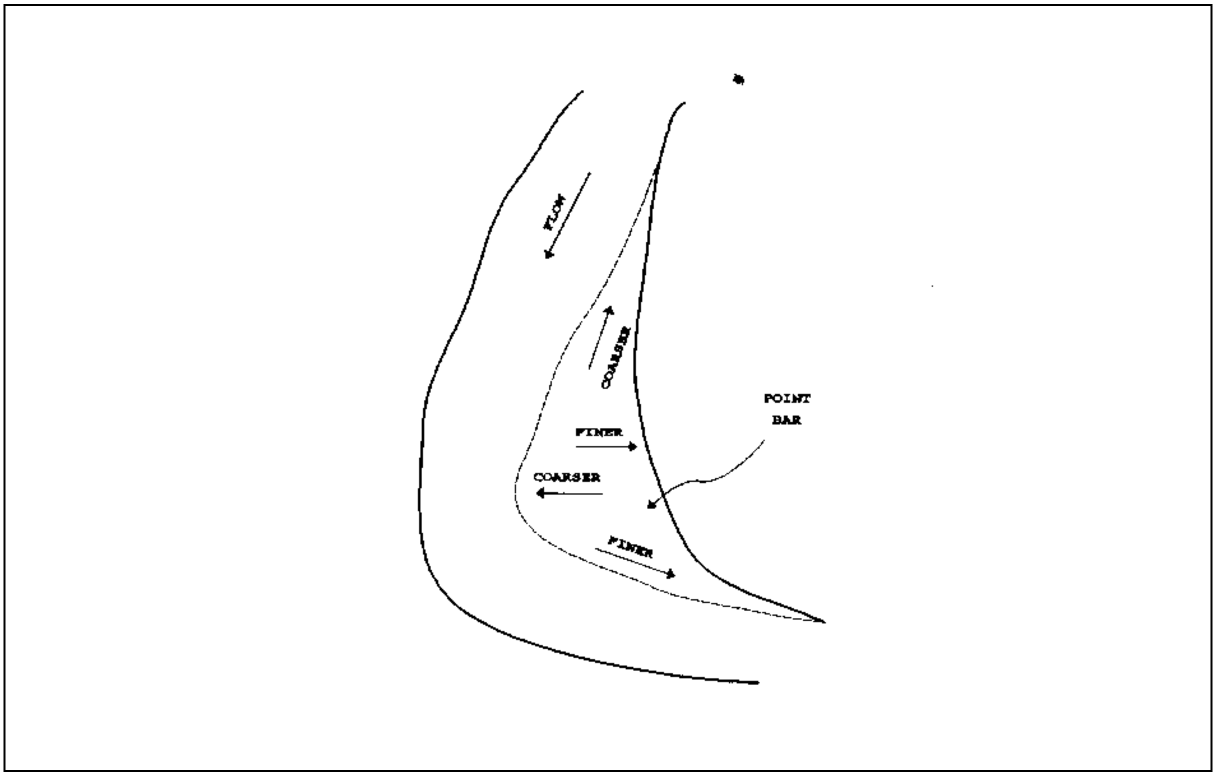


**Figure 7-10. Gradation pattern on a bar**

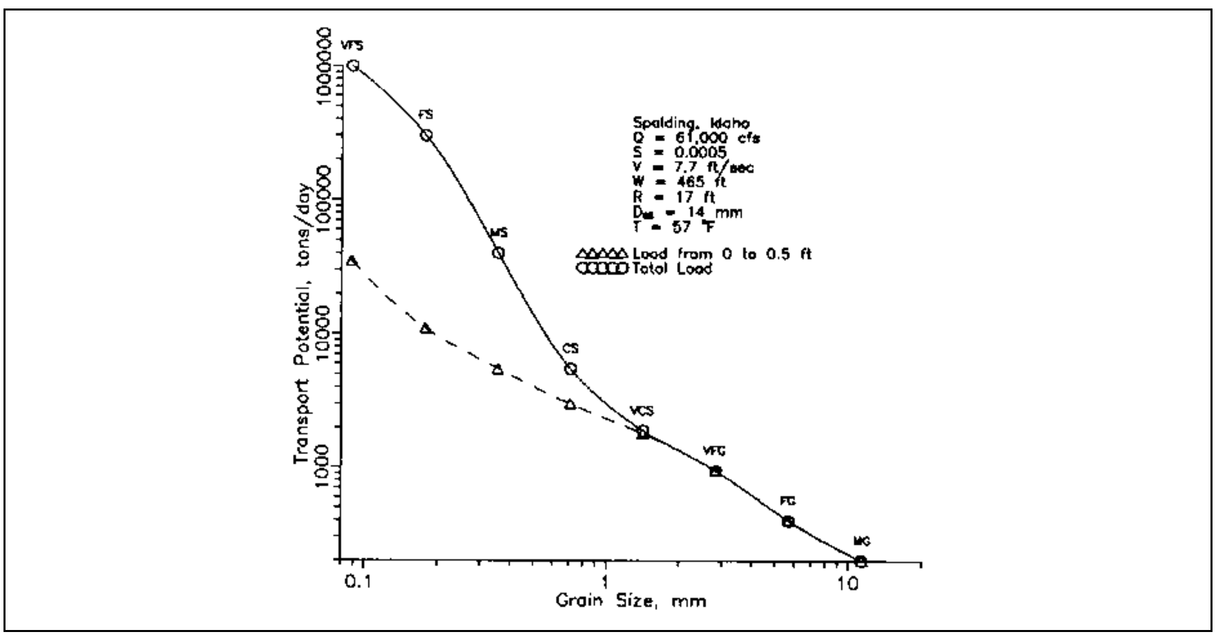


**Figure 7-11. Variation of sediment transport with grain size**

obtained from the NWS National Climatic Center at Asheville, North Carolina.

*d. Soil Conservation Service (SCS).* The local SCS office is a good point of contact for historic land use information, estimates of future land use, land surface erosion, and sediment yield. They have soil maps, ground cover maps, and aerial photographs which can be used as aids to estimate sediment yield. Input data for the Universal Soil Loss Equation is available for much of the United States. The SCS also updates reservoir sedimentation reports for hundreds of reservoirs throughout the country every 5 years, providing a valuable source of measured sediment data.

*e. Agricultural Stabilization and Conservation Service (ASCS).* This agency of the Department of Agriculture accumulates aerial photography of crop lands for allotment purposes. Those photographs include the streams crossing those lands and are therefore extremely valuable for establishing historical channel behavior because overflights are made periodically.

*f. Corps of Engineers.* Because the Corps gathers discharge data for operation of existing projects and for those being studied for possible construction, considerable data for a particular study area may already exist. The Corps has acquired considerable survey data, aerial and ground photography, and channel cross sections in connection with floodplain information studies. Corps laboratories have expertise and methods to assist in development of digital models.

*g. State agencies.* A number of states have climatologic, hydrologic, and sediment data collection programs. Topographic data, drainage areas, stream lengths, slopes, ground cover, travel, and times are often available.

*h. Local agencies, universities, consultants, businesses and residents.* Land use planning data can normally be obtained from local planning agencies. Cross section and topographic mapping data are also often available. Local agencies and local residents have in their verbal and photographic descriptions of changes in the area over time, information that is most valuable to the engineer. This source may include descriptions of channel changes associated with large flood events, incidents of caving banks, significant land use changes and when these changes occurred, records of channel clearing/dredging operations and other information. Newspapers and individuals who use rivers and streams for their livelihood are likewise valuable sources for data.

## 7-8. Data and Profile Accuracy

Agreement between calculated and measured water surface elevations of ± 0.5 foot is usually satisfactory for mobile boundary studies of natural rivers. Profiles of the computed average bed elevation may not correlate well with the prototype, but cross-sectional area changes should match prototype behavior.

*Section IV*
*Model Confirmation and Utilization*

## 7-9. Model Performance

Prior to using a numerical model for the analysis of a project, the model's performance needs to be confirmed. Ideally this consists of a split record test: selection (or calibration) of coefficients and verification of coefficients. The selection phase is intended to allow values for the coefficients to be chosen and adjusted so that the computed results reproduce field measurements within an acceptable error range. Computed results should be compared with measurements from the prototype to identify data deficiencies or physically unrealistic coefficients. Coefficients should then be adjusted as necessary, within the bounds associated with their uncertainty, to improve the agreement between observed and calculated values. Model adjustment does not imply the use of physically unrealistic coefficients to force a poorly conceived model to exactly match prototype measurements. If a discrepancy between model results and prototype data persists, then either there is something wrong with the model representation of the dominant physical processes (a model deficiency as a result of limiting assumptions), there is a deficiency in the representation of field data as model input (an application error), and/or there is something wrong with the measured data (a data deficiency). Therefore, if model calibration cannot be accomplished through the use of physically realistic values of the coefficients, the measured prototype data should be checked for possible errors and the numerical model (input data, basic equations, and solution algorithms) examined.

*a. Model adjustment.* Model adjustment is the process of data modification that produces simulation results that are in acceptable agreement with the prototype behavior. Adjustment consists of the selection of values for fixed and movable bed coefficients, and application of the art of transforming three-dimensional prototype measurements into "representative" one-dimensional data. Fixed bed coefficients are Manning's *n* values which do not depend on the characteristics of the movable boundary, coefficients of contraction and expansion, and ineffective flow area delineation. Movable bed coefficients are *n* values for the movable bed, which may depend on the rate of sediment transport. Development of representative data for one-dimensional computations is not done by simply averaging a collection of samples. For geometry, it is the selection of cross sections which will yield a one-dimensional approximation of hydraulic parameters that reconstitutes prototype values so that water and sediment movement in the model mimics that in the prototype. For sediment, it is the selection of bed sediment gradations, inflowing sediment loads and the fraction of sediment in each size class of those loads that reflect the dominant prototype processes.

(1) Manning's *n* values. The most credible method for determining *n* values for flood flows is to reconstitute measured high water profiles from historic floods. Another method is to reconstitute measured gage records. When there are no reliable field measurements the recourse is to use movable boundary roughness predictors for the movable bed portion of the cross section (Brownlie 1981, Limerinos 1970) and calibrated photographs (Barnes 1967, Chow 1959) for the overbank and fixed bed portions. Document prototyped conditions with photographs during the field reconnaissance.

(2) Contraction and expansion losses. Information on contraction and expansion losses is more sparse than for *n* values. King and Brater (1963) give values of 0.5 and 1.0, respectively, for a sudden change in area accompanied by sharp corners, and values of 0.05 and 0.10 for the most efficient transitions. Design values of 0.1 and 0.2 are suggested. They cite Hinds (1928) as their reference. Values often cited by the U.S. Army Corps of Engineers (1990b) are 0.1 and 0.3, contraction and expansion respectively, for gradual transitions.

(3) Representative data. Developing a one-dimensional representation of a three-dimensional open channel flow problem is an art. It requires one to visualize the three-dimensional flow lines in the actual problem and translate that image into a one-dimensional description. This step will often require several iterations to arrive at an acceptable representation. A useful approach is to "creep" up on a solution by first running a fixed bed simulation then adding sediment.

*b. Initial tests.*

(1) Steady flow, fixed-bed tests. Start with a steady state discharge of about bank-full. In a regime channel this is expected to be about the 2-year flood peak discharge. Ascertain that the model is producing acceptable hydraulic results by not only reconstituting the water surface profile, but also by plotting and examining the water velocity, depth, width and slope profiles. This test will often reveal width increases between cross sections that are greater than the expansion rate of the fluid and, therefore, require conveyance limits. Computed velocities at extremely deep bend sections may occasionally not be representative of sediment transport around the bend; one recourse is to eliminate those sections from the model. The results from running this discharge will also give some insight into how close the existing channel is to a "normal regime." That is, if there is overbank flow, justify that it does indeed occur in the prototype and is not just a "numerical problem" because in a regime channel the bank-full discharge is considered to be about the 2-year flood peak. It is useful to repeat this steady state, fixed bed, test for the maximum water discharge to be used in the project formulation before moving on to the movable bed tests. The key parameters to observe are water surface elevations, flow distribution between channel and overbanks, and velocities. Each study is unique, however, and one should regard the contents of this paragraph as suggestions that illustrate the analysis process and not a complete checklist.

(2) Steady flow, movable bed tests. It is useful to evaluate the model performance for the 2-year flood peak with a movable bed. Again, if the channel is near regime, this should be about a dominant discharge and result in very little aggradation or degradation. Before focusing on sediment transport, however, demonstrate that the Manning's *n* value for the channel is appropriate for a movable boundary. Make whatever adjustments are necessary to ensure that the *n* value for the stream bed portion of the cross section is in reasonable agreement with that obtained from bed roughness predictors. Also, the sediment transport rate will usually be higher at the beginning of the simulation than later because there is normally an abundance of fines in the bed samples which will be flushed out of the system as the bed layers are formed. A physical analogy is starting water to flow down a newly constructed ditch. It is important to balance the sizes in the inflowing bed material sediment load with transport potential and bed gradation. The scatter in measured data is usually sufficiently great to allow smoothing, but the adopted curves should remain within that scatter.

*c. Consequences of inaccurate n values.* In fixed bed hydraulics, a range of *n* values is typically chosen. The low end of that range provides velocities for riprap design, and the high end provides the water surface profile for flood protection. In movable bed studies such an approach is usually not satisfactory because of the feedback linkage between sediment transport and hydraulic roughness. Use of Manning's *n* values which do not conform with that linkage can result in either too much degradation or too much aggradation.

*d. Verification process.* The model adjustment process is to ensure that the model will reconstitute the trends which have been observed in the prototype. The second step, the verification process, is to change boundary conditions and rerun the model without changing the coefficients. This step establishes whether or not the coefficients which were selected in the first step will also describe the prototype behavior when applied to events not used in their selection. Change the inflowing sediment load as necessary to correspond with that during the time period selected for verification. Start with steady state data and progress to a hydrograph of flows.

(1) It is important to base the evaluation of model performance on those processes which will be used in decision making. These usually include the water surface profiles, flow distributions between channel and overbanks, water velocities, changes in cross-sectional area, sediment discharge passing each cross section, and accumulated sediment load by size class passing each cross section. A one-dimensional model may not precisely reconstitute thalweg elevations because the thalweg behavior is a three-dimensional process. Therefore, use cross-sectional end area changes or other measures rather than thalweg elevation in the verification test. Three types of graphs should be plotted to show verification results. The first is "variable versus elevation." An example, the comparison of calculated stages with the observed rating curve, is shown in Figure 7-12. The second graph is "variable versus distance" at a specific time as illustrated by the water surface and bed surface profiles in Figure 7-13. The third is "variable versus time" at selected cross sections along the study reach as shown in Figure 7-14.

(2) The verification period used may be several years long. If so, select only a few key values per year to plot. Plot the calculated water surface elevations at all gages in the study area as well as the observed elevations that occurred at the same time. Model performance may be quantified by computing the mean of the absolute values of error. Of course, the lower the mean value of

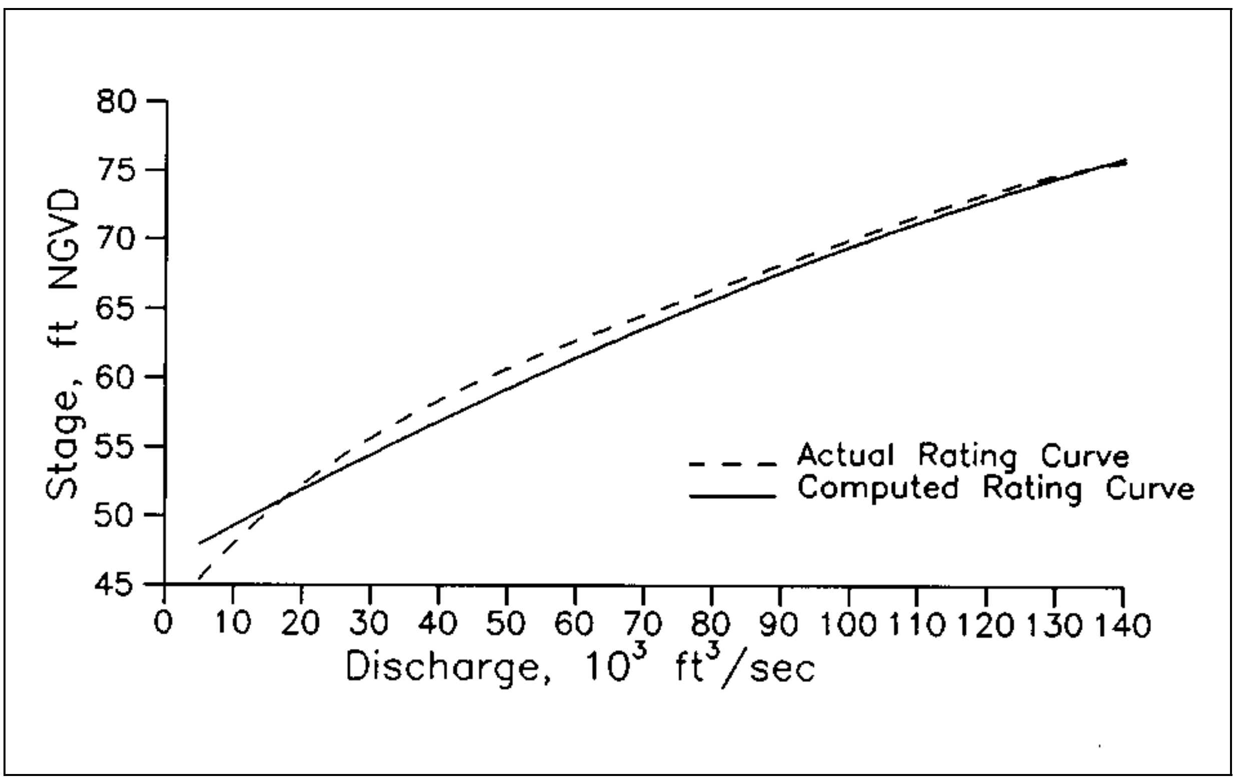


**Figure 7-12. Reconstituting the stage-discharge rating curve**

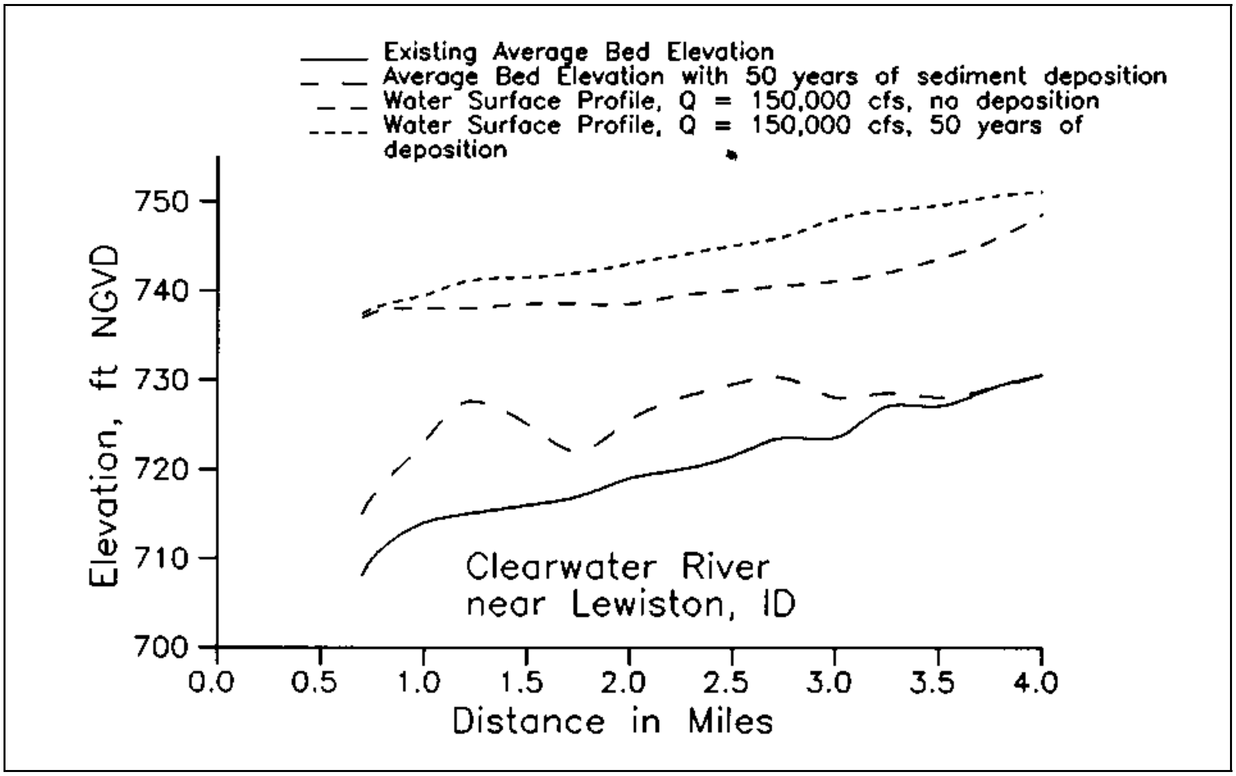


**Figure 7-13. Water surface and bed surface profiles**

error, the better the performance. Unfortunately, performance quality is defined by problem-specific characteristics and will probably differ from problem-to-problem. Good engineering judgment should be used to determine when the model's performance is satisfactory or requires additional adjustment.

*e. Correcting model performance.* If the calculated results do not follow the observed trends, take the following steps. First, plot the active bed gradation from cross sections at and downstream from inflow points using results from near the end of the hydrograph along with a bed gradation curve from field measurements. If the model is reproducing the dominant processes in the prototype, the key parameters should match reasonably well. The following suggestions illustrate the thought process that should occur when there is an unacceptable deviation.

(1) First, position the upstream boundary of the model in a reach of the river which is stable, and be sure the model exhibits that stability. That means that cross sections near the upstream end of the reach should neither significantly erode nor deposit. Attend to hydraulic problems starting at the downstream end and proceeding toward the upstream end of the model. Reverse that direction for sediment problems. Do not worry about scour or deposition at the downstream end of the model until it is demonstrating proper behavior upstream from that point.

(2) Second, be sure the model is numerically stable before adjusting any coefficients or processes.

(3) Once the above two conditions are met, focus attention on overall model performance. Check the boundary conditions to ascertain that the particle size classes in the inflowing sediment load have been assigned "representative" concentrations. Use depth and gradation of the bed sediment reservoir to determine that the model bed matches the prototype. Make plots for several different times because the gradation of the model bed will vary with the inflowing water-sediment mixture. Correct any inconsistencies in these data and try another execution. If any problem persists, check the field data for possible rock outcroppings and check calculated profiles for possible errors in nearby sections.

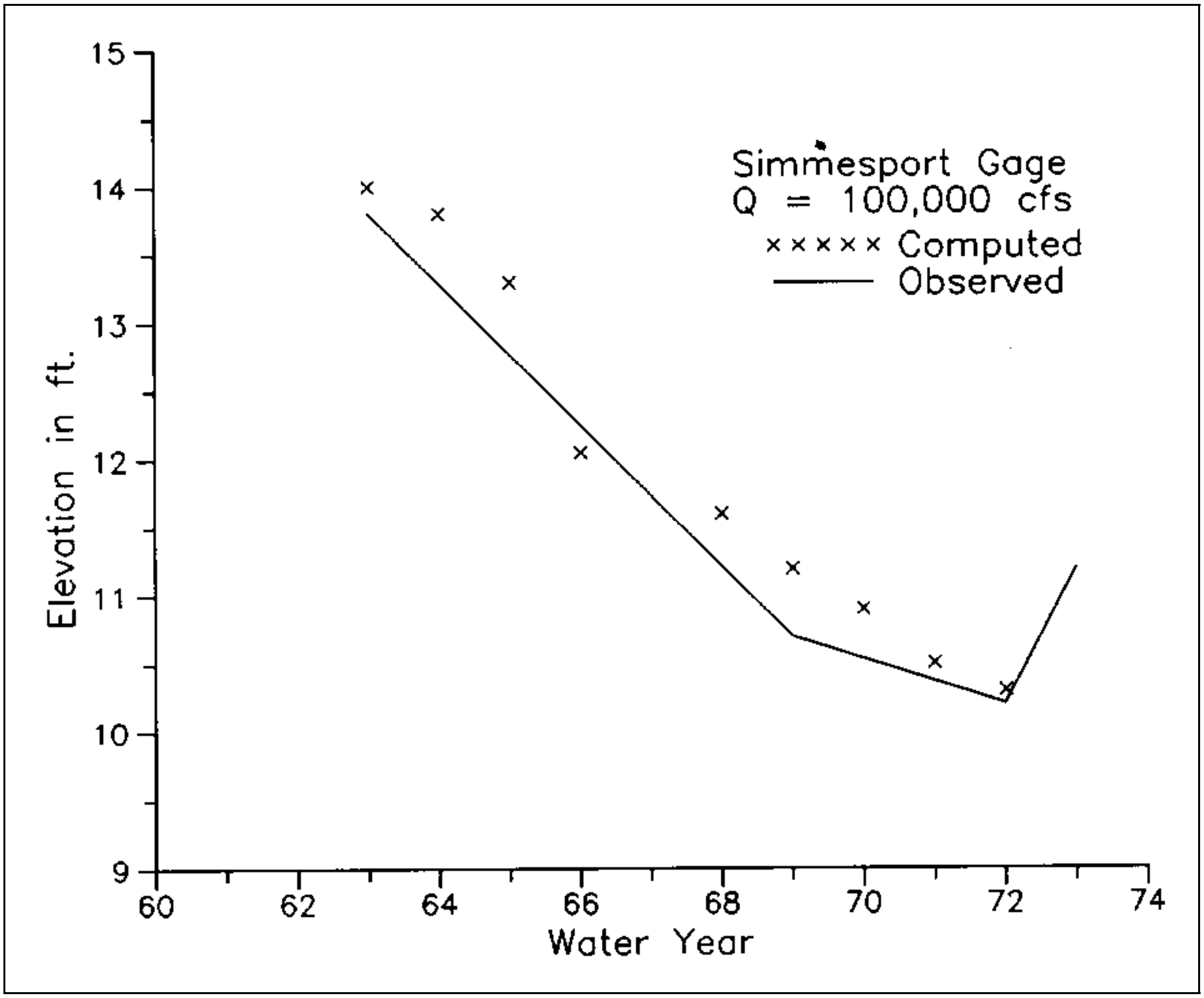


**Figure 7-14. Water surface trend plot (specific gage plot)**

(4) If calculated transport rates are too high, check prototype data for a gravel deposit which could be forming an armor layer.

(5) If calculated rates of deposition are too high or rates of erosion are too low, check top bank elevations and ineffective flow limits to ensure that the model is not allowing so much flow on the overbanks that the channel is becoming a sink.

(6) Finally, if none of the above actions produce acceptable performance, change the inflowing sediment load. First use a constant ratio to translate the curve without rotation. If that is not successful, rotate the curve within the scatter of data.

## 7-10. Development of Base Test and Analysis of Alternatives

The most appropriate use of a movable bed simulation is to compare an alternative plan of action with a base condition.

*a. The base test.* In most cases the base condition is the simulated behavior of the river under a "no action future." In a reservoir study, for example, the base test would calculate the behavior of the river, both upstream and downstream of the proposed dam site, without the dam in place. In many cases, the base test simulation should show little or no net scour or deposition. These are river reaches which are near equilibrium (where scour approximately equals deposition) under existing conditions.

*b. Plan tests.* The project alternatives can be simulated by modifying the base test data set appropriately. In the case of a reservoir, a dam can be simulated by inserting "operating rule data" into the base test model. For a channel improvement project, cross-sectional geometry and roughness can be changed. If a major change is to be analyzed, make the evaluation in steps. Avoid changing more than one parameter at a time because that makes the results difficult to interpret. For example, it is best to analyze a channel modification project in two steps. First, change the hydraulic roughness values and simulate future flows in the existing geometry. It will be necessary to select and justify the Manning's *n* for future conditions. Justify values by consideration of proposed design shapes, depths, channel lining materials, proposed vegetation on the overbanks, probable channel debris, anticipated riprap requirements, and maintenance agreements. Second, insert the modified cross sections and complete the analysis by simulating the alternatives to be tested. Also, select the appropriate contraction and expansion coefficients. Use model results as an aid in predicting future conditions; rely heavily on engineering judgment and look for anomalies in the calculated results. These "surprises" can be used by the experienced river engineer to locate data inadequacies and to better understand the behavior of the prototype system. Any unexpected response of the model should be justified very carefully before accepting the results.

*c. Presentation of results.* Results should be presented in terms of change from the base case wherever possible rather than absolute values. This will provide an assessment of the impacts of proposed projects.

*d. Sensitivity tests.* It is usually desirable during the course of a study to perform a sensitivity test. Quite often certain input data (such as inflowing sediment load) are not available, or subject to substantial measurement error. The impact of these uncertainties on model results can be studied by modifying the suspected input data by ± x percent and rerunning the simulation. If there is little change in the simulation, the uncertainty in the data is of no consequence. If large changes occur, however, the input data needs to be refined. Refinement should then proceed using good judgment and by modifying only one parameter or quantity at a time so as to be able to see the exact effect that overall changes may have. Sensitivity studies performed in this manner will provide sound insight into the prototype's behavior and lead to a sound model description of the real system.

*Section V*
*Computer Programs*

## 7-11. Introduction

Many computer programs are available for movable boundary simulations, and more will be created in the future. Two widely used programs are briefly discussed below as examples. This is not an exhaustive review. For any particular study, the need for use of a particular program or suite of programs must be defined and justified early in the study. See Chapter 3.

## 7-12. Scour and Deposition in Rivers and Reservoirs (HEC-6)

HEC-6 (U.S. Army Corps of Engineers 1991a) is a movable boundary model. It was formulated around Einstein's basic concepts of sediment transport; however, it is designed for the nonequilibrium case. Einstein did not address the nonequilibrium condition, but his "particle exchange" concept was extended in HEC-6 by noting that when sediment is in transport there will be a continual exchange between particles in motion and particles on the bed surface. The residue in the bed may be measurable, as in the case of the "bed material load", or it may be unmeasurable, as in the case of "wash load". The stability of particles on the bed surface may be related to inertia, as in the case of noncohesive particles; or that stability may be primarily electrochemical, as in the case of cohesive particles. Energy forces acting to entrain a particle may be primarily gravity induced, as in the case of flow in inland rivers; or the forces may be combinations of energy sources such as gravity, tides, waves, and density currents, as in the coastal zone. Different types of sediment require different entrainment functions depending upon the propensity of the sediment to change hydrodynamic and physical properties of the flow and upon the sensitivity of the sediment type to water temperature and chemistry.

*a. Equations of flow.* The equations for conservation of energy and water mass are simplified by eliminating the time derivative from the motion equation which leaves the gradually varied steady flow equation. It is solved using the standard step method for water surface profiles. The following terms are included:

$$\frac{\partial h}{\partial x} + \frac{\partial(\alpha U^2/2g)}{\partial x} = S_e$$ (conservation of energy) (7-2)

where

$g$ = acceleration due to gravity
$h$ = water surface elevation
$S_e$ = slope of energy line
$U$ = flow velocity
$x$ = distance in the direction of flow
$\alpha$ = correction for transverse distribution of flow velocity

$$Q = UA + Q_1$$ (conservation of water) (7-3)

where

$A$ = cross-sectional area of flow
$Q_1$ = lateral or tributary inflow
$Q$ = main stem water discharge downstream from $Q_1$
$U$ = main stem mean water velocity upstream from $Q_1$

*b. Friction and form losses.* Both friction and form losses are included in $S_e$; bed roughness is prescribed with Manning $n$ values. $n$ values may vary with water discharge, location, or be related to bed material size (Limerinos 1970).

*c. Equation of sediment continuity.* The Exner equation is used for conservation of sediment:

$$\frac{\partial Q_s}{\partial x} + B_s\frac{\partial Y_s}{\partial t} + q_s = 0$$ (conservation of sediment) (7-4)

where

$B_s$ = width of bed sediment control volume
$Q_s$ = volumetric sediment discharge rate
$q_s$ = lateral or tributary sediment discharge rate
$t$ = time
$Y_s$ = bed surface elevation

*d. Computational methodology.* Descriptions of the computational methodology used in HEC-6 and application of the program are presented in HEC by the U.S. Army Corps of Engineers (1991a).

## 7-13. Open Channel Flow and Sedimentation (TABS-2)

*a. Purpose.* The purpose of the TABS-2 system (Thomas and McAnally 1985) is to provide a complete set of generalized computer programs for two-dimensional numerical modeling of open-channel flow, transport processes, and sedimentation. These processes are modeled to help analyze hydraulic engineering and environmental conditions in waterways. The system is designed to be used by engineers and scientists who need not be computer experts.

*b. Description.* TABS-2 is a collection of generalized computer programs and utility codes integrated into a numerical modeling system for studying two-dimensional hydraulics, transport, and sedimentation processes in rivers, reservoirs, bays, and estuaries. A schematic representation of the system is shown in Figure 7-15.

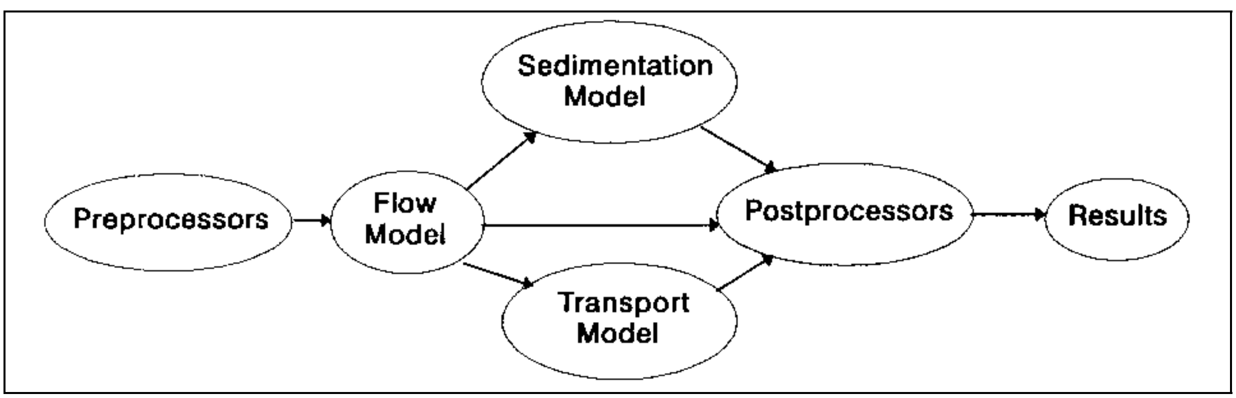


**Figure 7-15. TABS-2 schematic**

*c. Uses.* It can be used either as a stand-alone solution technique or as a step in the hybrid modeling approach. The basic concept is to calculate water-surface elevations, current patterns, dispersive transport, sediment erosion, transport and deposition, resulting bed surface elevations, and feedback to hydraulics. Existing and proposed geometry can be analyzed to determine the impact of project designs on flows, sedimentation, and salinity. The calculated velocity pattern around structures and islands is particularly useful. Some applications of TABS-2 are referenced in Chapter 3.